October 10, 2012

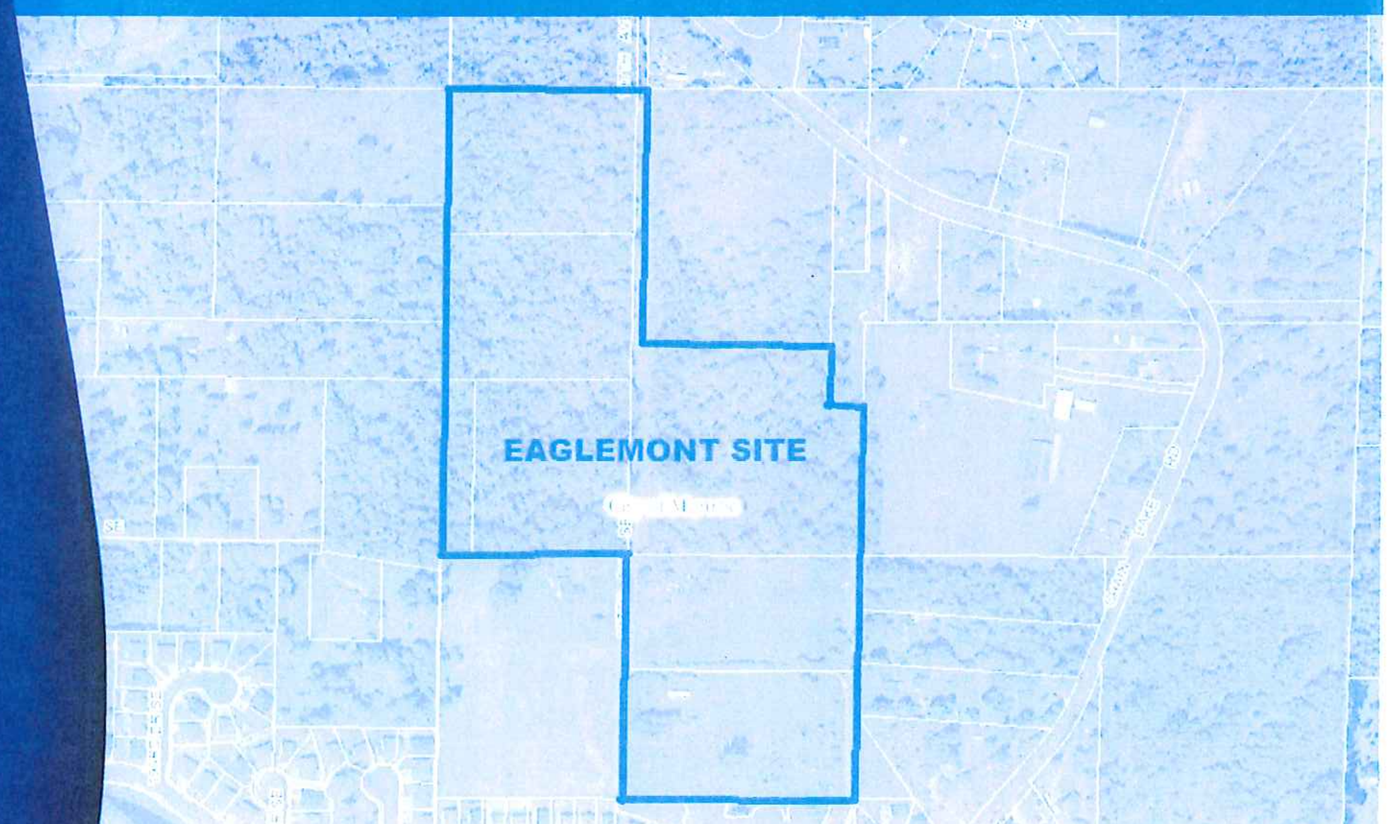
Prepared for:
RAD Development
16531 13th Avenue W. Suite #A107
Lynnwood, WA 98037
(206) 299-2600

Eaglemont

Technical Information Report

Project Location:
13611 197th Ave SE
Monroe, WA 98272

SDA Project #278-003-12



SDA
CIVIL ENGINEERING • PROJECT MANAGEMENT • PLANNING • SURVEYING
1704 W. Marine View Drive, Suite 140 Everett, WA 98201 • 425.486.6533 • www.sdaengineers.com

EXHIBIT# M5

Eaglemont
Technical Information Report (TIR)

Project Location:
13611 197th Ave Se
Monroe, Washington 98272

Prepared For:
RAD Development
16531 13th Avenue W., Suite #A107
Lynnwood, Washington 98037
(206) 299-2600

Prepared By:
Site Development Associates
1724 West Marine View Dr., Ste. 140
Everett, WA 98201
(425) 486-6533

Date:
October 10, 2012

Project Number:
278-003-12

ANDREW C. REEVES
STATE OF WASHINGTON
31257
REGISTERED
PROFESSIONAL ENGINEER

10.10.12
EXP. 6.17.13

TABLE OF CONTENTS

SECTION 1

EXECUTIVE SUMMARY

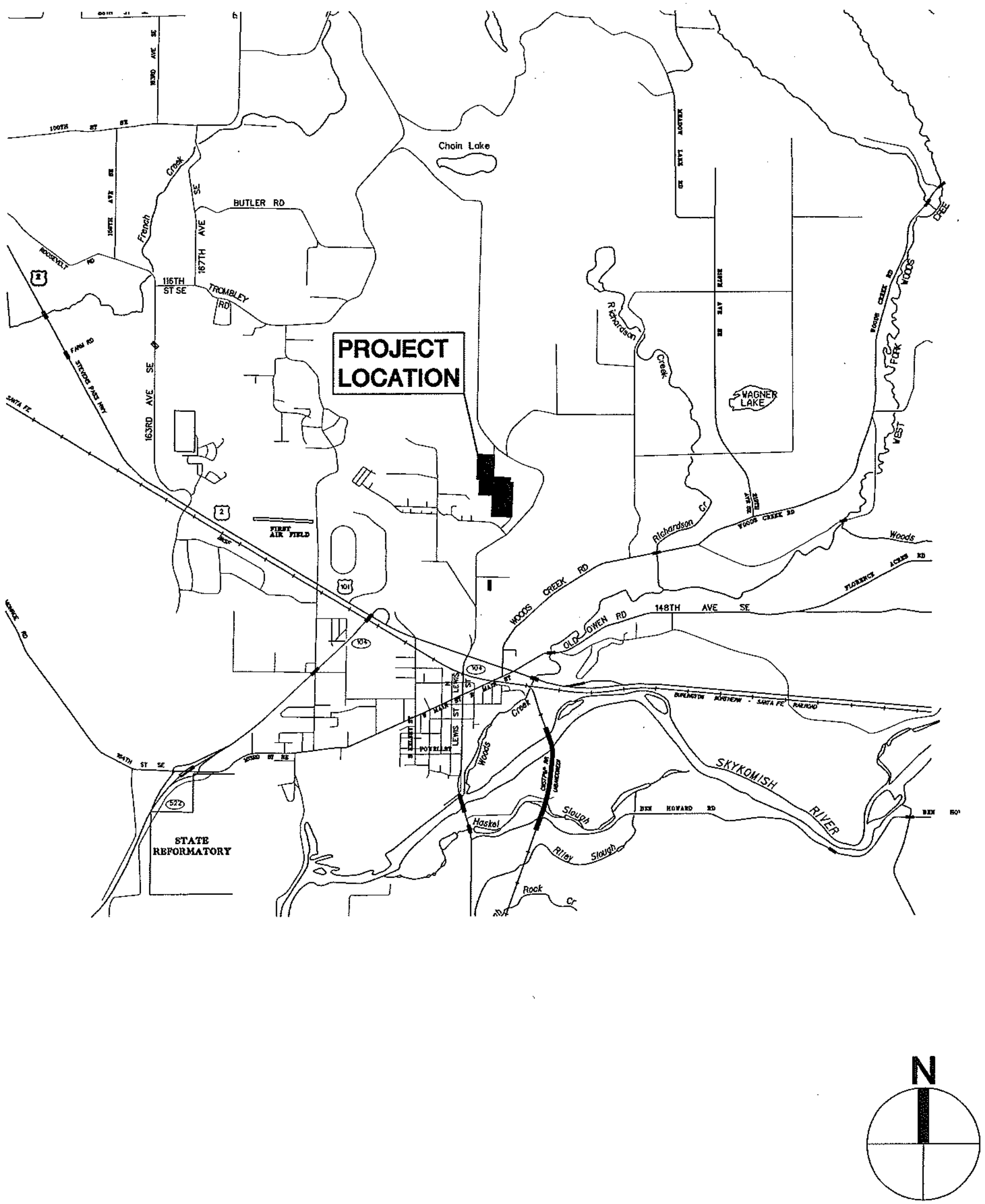
PROJECT OVERVIEW

The proposed project is to construct a subdivision of approximately 35 acres into 146 new single family residences, located at the existing terminus of 199th Avenue SE in the subdivision of Sinclair Heights in the City of Monroe. The project will clear, grade and construct roads, utility extensions and features and eventually single family residences on the lots. There is currently one building with associated driveways, which will be removed. The neighboring plat of "Sinclair Heights" has provided sewer, water, drainage, and dry utility stubs in the adjacent public road terminus called 199th Avenue SE. These stubs will be utilized in the construction of the project. The project is located in the east half of the SW ¼ of the NW ¼ of section 31, Township 28 North, Range 7 East, W.M. More specifically, the project occupies tax lot numbers 28073100201000, 28073100203300, 28073100203400, 28073100200l100, 28073100204000, 01010300050200, and 0101030050100. A vicinity map has been included as **Figure 1** of this document.

The site is currently mostly forested and wooded with some pasturelands in the southern portions of the site. There is a utility easement in the middle of the site on a N-S bearing that has a gravel maintenance road within it. This gravel road connects to chain lake road to the north. The site has two drainage basins, one that drains to the south toward the Sinclair Heights project, and one to the north that drains overland to the north, towards Chain Lake Road. The south basin that contains the vast majority of the site, will contain a large detention pond. This pond will be located at the south end of the site and will be made completely of earthen berms and cut slopes. The pond will have 1' of dead storage for sediment removal and a biofiltration swale downstream of the detention pond. The biofiltration swale will discharge to a level spreader which will disperse flows into the adjacent wetland to the south of the site. The pond will be fitted with an emergency overflow structure, or "Bird Cage" that will be fitted on the frop T orifice release structure, and then a secondary emergency overflow spillway over the south bank of the detention pond. This secondary emergency overflow will be armored with quarry spalls and will also drain south into the adjacent wetland. The detention pond has been designed utilizing the latest version of WWHM3 continuous storm modeling software as per the 2005 DOE manual for existing versus proposed drainage release rates. The point of compliance is the location where the flows leave the proposed level spreader, which is the southernmost portion, and the point of the lowest elevation of the site.

The northern basin which is a very small portion of the site (3.6 acres of the total 35 acres), will be released to the north in its natural drainage course toward Chain Lake Road. Of the developed portion of the north basin, only the downhill 0.83 acres will be released to the north. The remainder of the plat in the north basin (2.77 acres) will be diverted to the south basin and into the proposed detention pond. This is due to the fact that the several utility (natural gas, domestic water) easements within this north basin make it very difficult to design a detention system within this north basin. And by over detaining in the south basin, within the existing detention pond, we are able to eliminate the need for two detention systems. Thus providing a more cost efficient storm drainage system, with much less maintenance for the city of Monroe and the homeowners association to operate and maintain. This 0.83 acres was chosen to keep the developed release rates .vs. pre-developed rates to the north basin within the guidelines of the 2005 DOE manual, thus meeting all Point of Compliance (POC) release rate criteria for the entire site, while utilizing one detention pond.

Site Soils below the topsoil layer consist of Vashon Lodgment Till. This material is an unsorted mixture of loose to medium dense, reddish brown to tan silty sand with gravel and scattered cobbles and boulders. Below depth ranging from approximately 2-4 feet, these sediments became dense to very dense and grayish tan. The Vashon lodgment till consists of an unsorted mixture of silt, sand and gravel that was deposited directly from basal, debris laden glacial ice during the Vashon Stade of the Fraser Glaciation, approximately 12,500 to 15,000 years ago. The high relative density characteristic of the lodgment till is due to its consolidation by the massive weight of ice from which it was deposited. These deposits are generally dense to very dense and are of extremely low permeability in their native undisturbed state.



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ARC Design SJR Drawn 10/11/12 Date 278-003-12 Project No.	SDA Civil Engineering Project Management Planning 1724 W. Marine View Drive, Suite 140; Everett, Wasington 98201 Office: 425.486.6533 Fax: 425.486.6593 www.sdaengineers.com	EAGLEMONT VICINITY MAP	NTS Scale 1 Figure No.

SECTION 2

MINIMUM REQUIREMENT #1

PREPARATION OF STORMWATER SITE PLANS

STORMWATER SITE PLANNING PROCESS

The City of Monroe has adopted the 2005 Washington State Department of Ecology Stormwater management Manual for the Puget Sound Basin as the governing design document for surface runoff control. The following is a listing of the applicable minimum "core" and "special" requirements outlined in Chapter 1 of the manual, with a brief description of how each was addressed:

- ***Step 1: Collect and Analyze Information on Existing Conditions***

  Runoff can be expected to follow the existing ground topography, and flow in a southeastern direction for the south basin, and a northwestern direction for the north basin. As site slopes in the project clearing area are flat to moderate(0%-15%), and the are to be cleared is large with long reaches of drainage courses, there is medium to high potential for erosion. This can be easily controlled with erosion control measures, as slopes are very consistent.

  Site Soils below the topsoil layer consist of Vashon Lodgment Till. This material is an unsorted mixture of loose to medium dense, reddish brown to tan silty sand with gravel and scattered cobbles and boulders. Below depth ranging from approximately 2-4 feet, these sediments became dense to very dense and grayish tan. The Vashon lodgment till consists of an unsorted mixture of silt, sand and gravel that was deposited directly from basal, debris laden glacial ice during the Vashon Stade of the Fraser Glaciation, approximately 12,500 to 15,000 years ago. The high relative density characteristic of the lodgment till is due to its consolidation by the massive weight of ice from which it was deposited. These deposits are generally dense to very dense and are of extremely low permeability in their native undisturbed state.

- ***Step 2: Prepare a Preliminary Development Layout***

  The layout for the site is controlled primarily by the on-site utility easements and the exterior boundaries of the parcels, there are no site wetlands. The Project drainage is in two basins, with the south basin flowing into the project's large detention pond and then to the adjacent plat drainage system, and the north basin flowing to the north at or below pre-existing flow rates per the 2005 DOE manual. Site access is limited to the one public road that connects to the parcel, and a secondary easement road that connects the site to Chain Lake Road to the North.

- ***Step 3: Perform Offsite (Upstream and Downstream) Analysis***

  There are some small upstream basins to the site to the east and west that flow overland onto the site. They are shown in Appendix 2-A. As they are small in nature, we will allow the onsite drainage system to capture the sheet flow from the adjacent properties and allow it to be routed thru our drainage system. The South basin upstream basin is 1.89 acres in size, and the north basin upstream is 0.49 acres. These areas will simply be added in both the existing and mitigated basins as forested, thus the detained volume will be unchanged.

  The two downstream drainage courses (north and south) are similar in nature. The south Basin downstream is thru the adjacent plat of Sinclair Heights and is almost completely in pipes and open ditches. The north downstream basin flows to the Chain Lake Roadside ditch, then under Chain lake road to an adjacent wetland, and then to the north and west. Both are analyzed in detail below, with exhibits in the appendix.

  South Basin:

  The downstream flow from the project starts in the adjacent wetland, tract 996, of the adjacent Sinclair Heights Subdivision. Flows continue SE in the wetland to an 18" ductile iron culvert under 199th avenue (Photos 1 and 2) to the wetland in Tract 997, flows then continue due south in the wetland where they enter another 18" ductile Iron culvert under Rainier Road NE (Photos 3 and 4), and into another small wetland. After flowing sw in the wetland flows enter an 18" concrete culvert that goes under the walkway for Sinclair Heights (photos 5 and 6) along the south property line. After this, flows continue sw to a private 18" culvert to the chain lake r/w. From here flows travel in the se direction along the n side of Chain lake Road in a series of 18" pipes and roadside ditches until they turn due west into a large wetland. (photos 7-12) Flows continue to the west along the north side of the church and eventually enter the lakeside parcel, flow south under highway 2 and eventually into the Snohomish river.

Photo #1, pipe out of tract 996

Photo #2, 18" ductile out of tract 996, into.997 (under 199th ave)

Photo #4, 18" ductile iron out of tract 997 (Under Rainier Road)


Photo #5, outfall of pipe under Rainier Road


Photo #6, Inlet of 18" HDPE under Sinclair Heights Walkway


Photo #7, outlet of 18" HDPE under Sinclair Heights Walkway


Photo #8, outlet of 18" HDPE culvert on private property between Sinclair Heights and Chain Lake Rd. R/W

Photos of pipes and ditch along N side of Chain lake Road, Top left, followed by top left followed by bottom lt and finally bottom rt.

Photos of pipes and ditch along N side of Chain lake Road, Top left, followed by top left followed by bottom lt and finally bottom rt.

North Basin:

Flows from the north basin will flow to the north to the Chain Lake Road Ditch. This path will be almost entilrely sheet flow except for the proposed road connection with Chain Lake Road.

- ***Step 4: Determine Applicable Minimum Requirements***

  As the site is 35 acres and is proposing 146 lots, all 10 minimum requirements apply.

- ***Step 5: Prepare a Permanent Stormwater Control Plan***

  The site is currently mostly forested and wooded with some pasturelands in the southern portions of the site. There is a utility easement in the middle of the site on a N-S bearing that has a gravel maintenance road within it. This gravel road connects to chain lake road to the north. The site has two drainage basins, one that drains to the south toward the Sinclair Heights project, and one to the north that drains overland to the north, towards Chain Lake Road. The south basin that contains the vast majority of the site, will contain a large detention pond. This pond will be located at the south end of the site and will be made completely of earthen berms and cut slopes. The pond will have 1' of dead storage for sediment removal and a biofiltration swale downstream of the detention pond. The biofiltration swale will discharge to a level spreader which will disperse flows into the adjacent wetland to the south of the site. The pond will be fitted with an emergency overflow structure, or "Bird Cage" that will be fitted on the frop T orifice release structure, and then a secondary emergency overflow spillway over the south bank of the detention pond. This secondary emergency overflow will be armored with quarry spalls and will also drain south into the adjacent wetland. The detention pond has been designed utilizing the latest version of WWHM3 continuous storm modeling software as per the 2005 DOE manual for existing versus proposed drainage release rates. The point of compliance is the location where the flows leave the proposed level spreader, which is the southernmost portion, and the point of the lowest elevation of the site.

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- ***Step 6: Prepare a Stormwater Pollution Prevention Plan (SWPPP)***

  The 12 step outline is included in section 3 of this report, the full SWPPP is included as Appendix 3-A.

- ***Step 7: Complete the Stormwater Site Plan***

The stormwater site plan will be very similar to the plan developed at the preliminary stage sof the project, as outlined above in step 5..

Conveyance System

A full conveyance analysis for the plat will be performed at construction review.

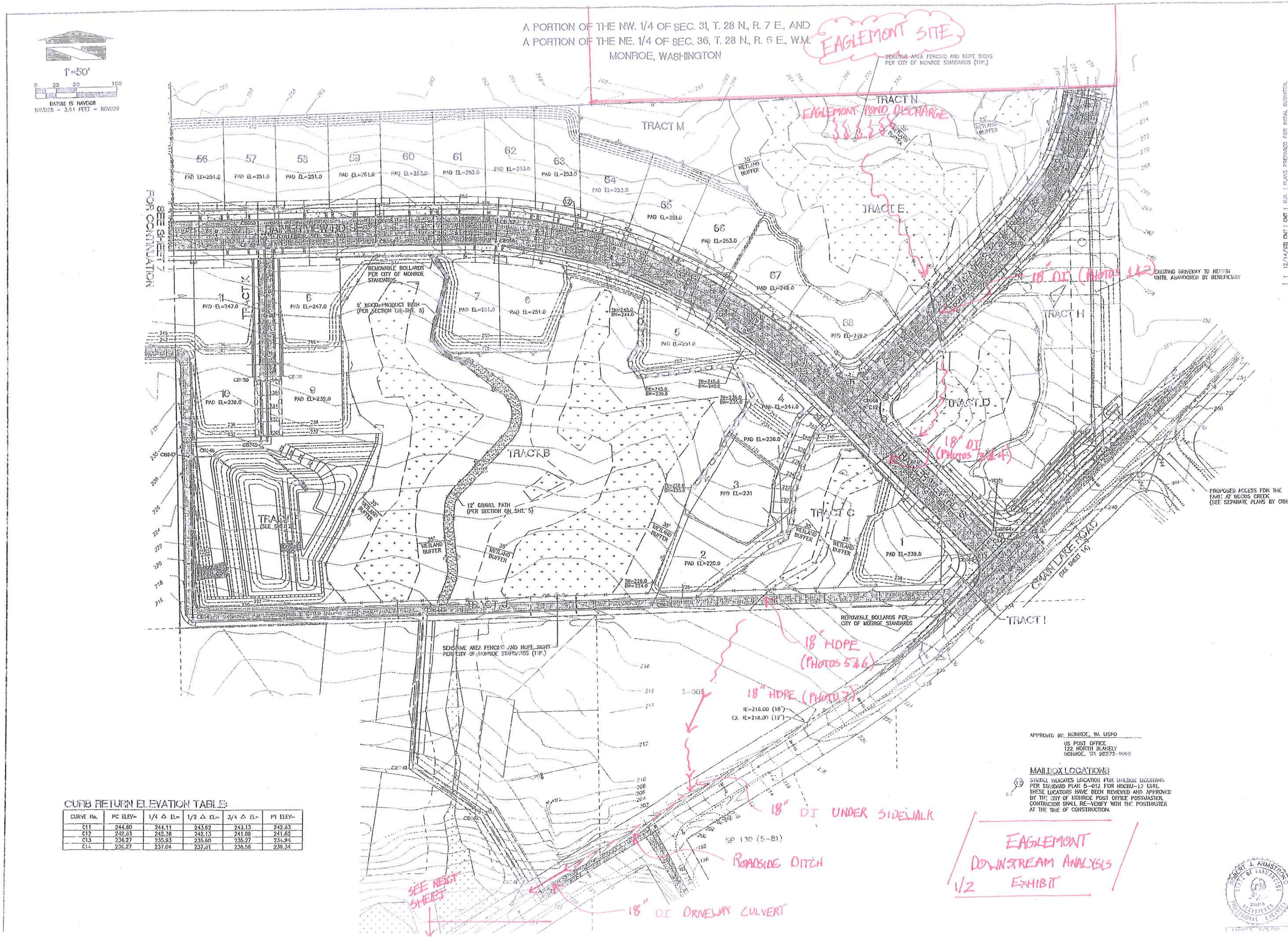
APPENDIX 2-A

UPSTREAM ANALYSIS EXHIBIT

APPENDIX 2-B

DOWNSTREAM ANALYSIS EXHIBIT
(South Basin)

A PORTION OF THE NW. 1/4 OF SEC. 31, T. 28 N., R. 7 E., AND
A PORTION OF THE NE. 1/4 OF SEC. 36, T. 28 N., R. 6 E., W.M.
MONROE, WASHINGTON



CURB RETURN ELEVATION TABLE:

CURVE No.	PC ELEV=	1/4 △ EL=	1/2 △ EL=	3/4 △ EL=	PT ELEV=
C11	244.60	244.11	243.62	243.13	242.63
C12	242.63	242.38	242.13	241.88	241.62
C13	236.27	235.93	235.60	235.27	234.94
C14	236.27	237.04	237.81	238.58	239.34

No.	Date	By	Ckd.	Appr.	Revision
1	12/14/02	CMT	CMT	RJA	PLANS PREPARED FOR INITIAL SUBMITTAL

Title: ROAD, GRADING AND DRAINAGE PLAN
EAST HALF
SINCLAIR HEIGHTS

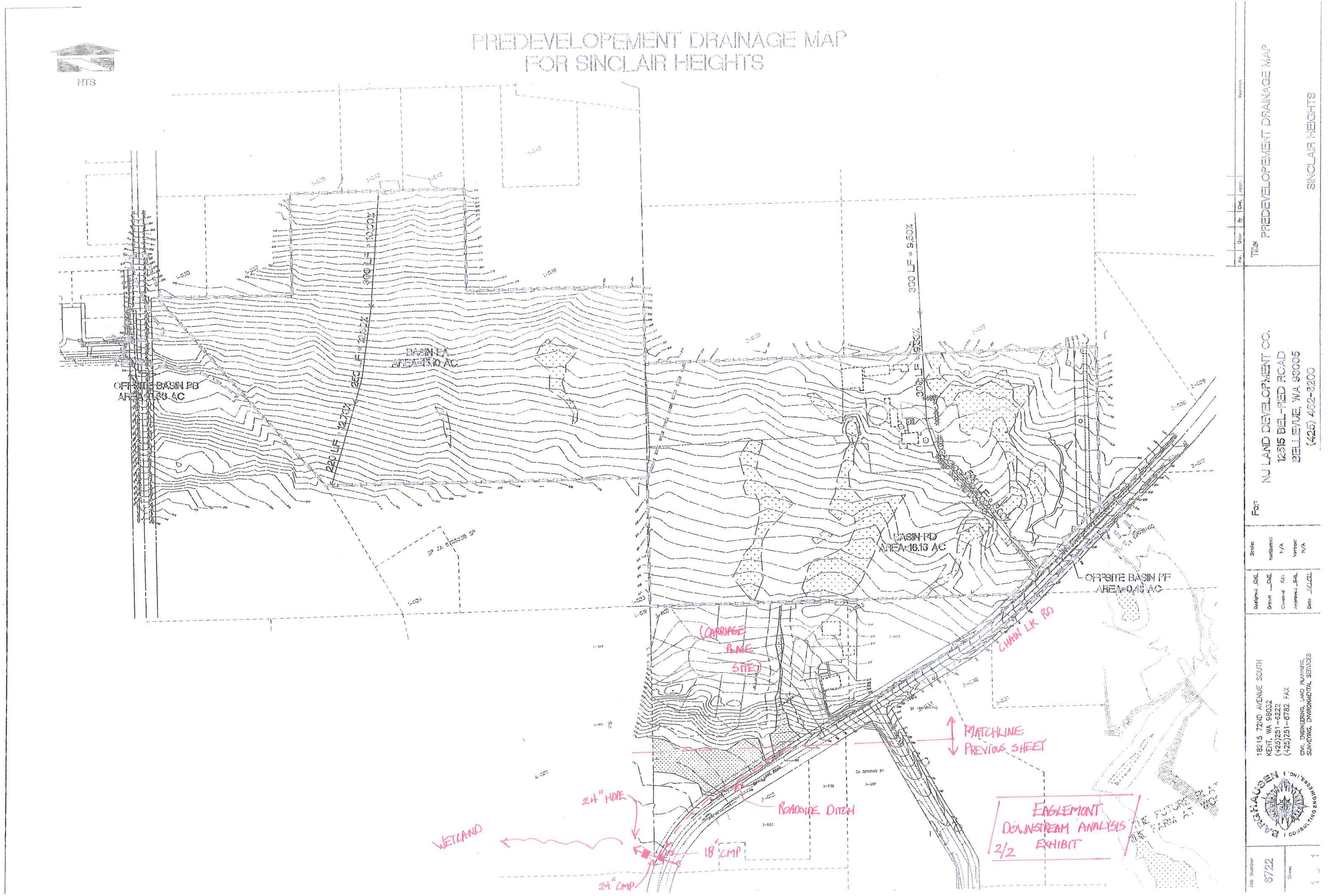
For: NJ LAND DEVELOPMENT COMPANY
12515 BEL-RED ROAD, SUITE 204
BELLEVUE, WA 98005
CONTACT: DEREK SINCLAIR
(425) 462-8200 ext. 228

Designed CMT · Drawn CMT · Checked RJA · Approved RJA · Date 6/4/01
Scale: Horizontal 1"=50' · Vertical N/A

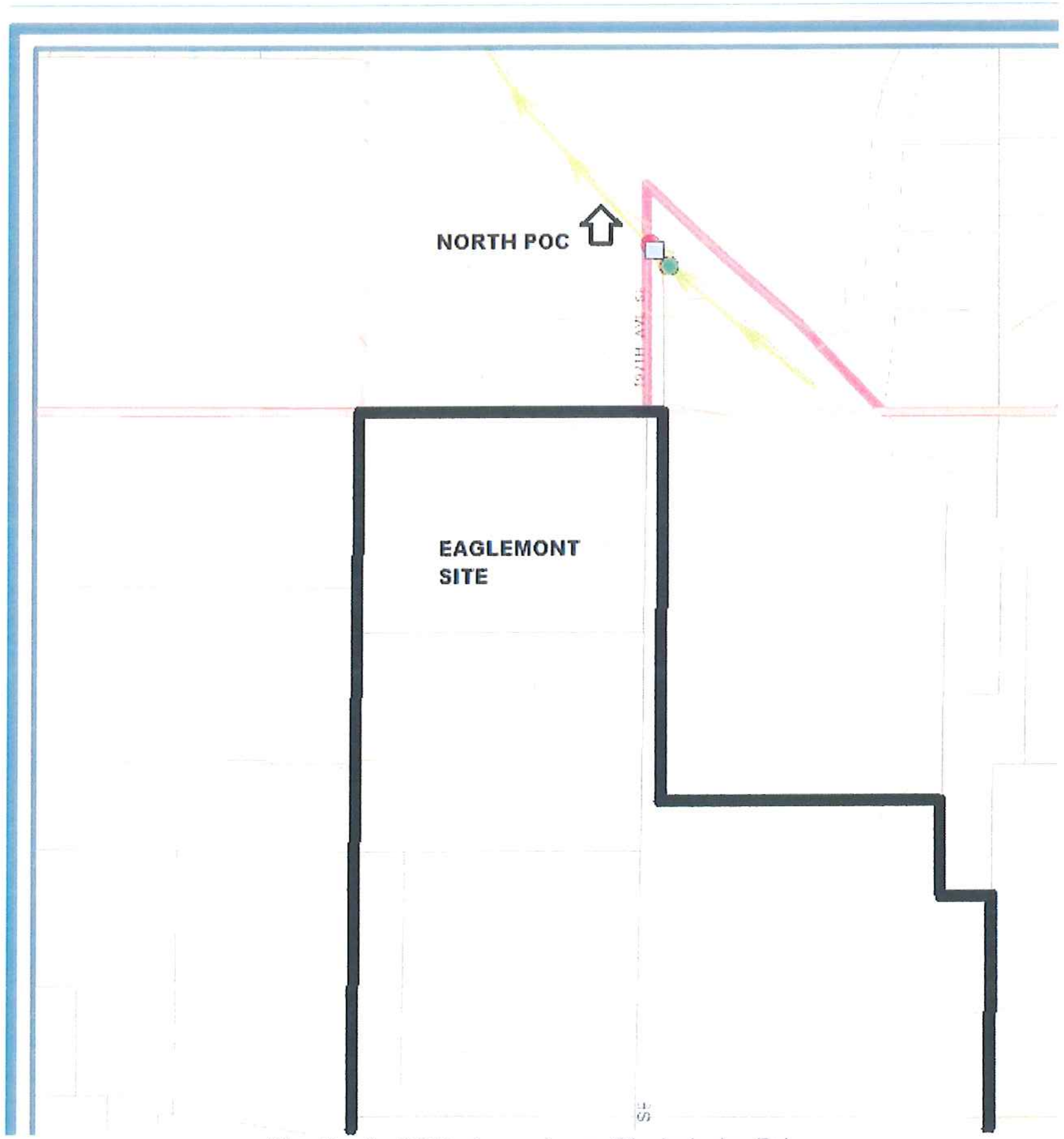
18215 72ND AVENUE SOUTH
KENT, WA 98032
(425)251-6222
(425)251-8782 FAX
CIVIL ENGINEERING, LAND PLANNING, SURVEYING, ENVIRONMENTAL SERVICES

ROBERT J. ARMSTRONG · STATE OF WASHINGTON · REGISTERED PROFESSIONAL ENGINEER · 25018

Job Number 5722


PREDEVELOPEMENT DRAINAGE MAP
FOR SINCLAIR HEIGHTS
NTS
BASIN PA
AREA=15.10 AC
OFFSITE BASIN PB
300 LF @ 10.00%
220 LF @ 12.73%
300 LF @ 9.50%
BASIN PD
AREA=16.16 AC
OFFSITE BASIN PF
AREA=0.48 AC
(CARRIAGE PLACE SITE)
CHAIN LK RD
MATCHLINE PREVIOUS SHEET
ROADSIDE DITCH
24" HDPE
WETLAND
18" CMP
24" CMP
EAGLEMONT DOWNSTREAM ANALYSIS EXHIBIT
2/2
PREDEVELOPEMENT DRAINAGE MAP
SINCLAIR HEIGHTS
NJ LAND DEVELOPMENT CO.
12315 BEL-RED ROAD
BELLEVUE, WA 98005
(425) 462-3200
18215 72ND AVENUE SOUTH
KENT, WA 98032
(425)251-6222
(425)251-6782 FAX
CIVIL ENGINEERING, LAND PLANNING, SURVEYING, ENVIRONMENTAL SERVICES
CONSULTING ENGINEERS, INC.
Job Number
6722

DOWNSTREAM ANALYSIS EXHIBIT
(North Basin)



The North POC shown is on Chain Lake Rd.

This is the last section of the North Basin ¼ mile downstream analysis

APPENDIX 2-C

CONVEYANCE SYSTEM DESIGN CALCULATIONS

SECTION 3

MINIMUM REQUIREMENT #2

CONSTRUCTION STORMWATER POLLUTION PREVENTION (SWPPP)

A Stormwater Pollution Prevention Plan (SWPPP) is required to address 12 specific pollution prevention elements per SCC 30.63A. These elements are listed and summarily addressed below, the full SWPPP is included as appendix 3-A and is included in this report, but bound separately for convenience in the field:

1. Mark Clearing Limits

Clearing limits will be flagged or fenced by the contractor or project surveyor prior to commencement of construction activity.

2. Establish Construction Access

A stabilized rock construction entrance will be installed at the entrance to the plat at the onset of construction.

3. Detain Flows

Prior to significant clearing, a permanent detention facility shall be constructed, so that it can be used for temporary sediment control. A temporary sediment riser shall be installed in order to ensure proper sediment control. Once the facility is constructed, the site shall be cleared and graded, and all surface water controls shall direct runoff to this facility. When final grading is complete and the site is stabilized, the temporary sediment riser shall be replaced with a permanent flow control structure.

4. Install Sediment Controls

Filter fabric fencing (silt fence) shall be installed around the downstream perimeter of the site in order to keep sediment-laden stormwater from leaving the site. The fencing shall be inspected periodically to ensure its continued effectiveness.

5. Stabilize Soils

Exposed soils shall be stabilized through mulching or hydroseeding when the not actively worked for a significant period of time. Permanent vegetation shall be established through hydroseeding once the site has reached final grade.

6. Protect Slopes

The project calls for the installation of rockeries and retaining walls. The faces of these walls shall be protected until the facing stones or rocks are installed. No other significant slopes are proposed.

7. Protect Drain Inlets

The temporary erosion and sediment control plan calls for a filter fabric sock to be installed at all nearby catch basin inlets. Filter fabric protection shall be placed in all new catch basins as they are installed.

8. Stabilize Channels and Outlets

All temporary interceptor swales shall contain check dams whenever a drop of 2 vertical feet occurs. Water discharged from the sedimentation facility shall outfall onto a rip-rap splash pad or level spreader.

9. Control Pollutants

All waste materials shall be disposed of in an approved location, in accordance with City of Monroe Standards. In order to reasonably prevent a contamination event (such as a fuel spill), all major vehicle maintenance shall occur off-site to the greatest extent practicable. The contractor shall provide a vehicle staging area near the entrance to the site where all fueling and maintenance activity is likely to take place. This is intended to contain the area in which a contamination event is likely to take place. The contractor shall immediately contain and clean-up an area in which a contamination event occurs.

10. Control De-Watering

No significant dewatering is expected to occur during this project.

11. Maintain BMPs

All BMPs should be monitored and maintained regularly to ensure adequate operation. A TESC supervisor shall be identified at the beginning of the project to provide monitoring and direct the appropriate maintenance activity. As site conditions change, all BMPs shall be updated as necessary to maintain compliance with City standards.

12. Manage the Project

The project will begin with a pre-construction conference in which an on-site TESC supervisor shall be identified. The on-site supervisor shall monitor all TESC facilities regularly and maintain a log of inspections and improvements to demonstrate compliance with City standards. The project erosion control should be phased if the weather forecast is not solid. Thus the site is cleared, stabilized with TESC measures, and the moved on to the next phase. It will be important that the entire site is in conformance with City of Monroe erosion control standards at all times. The TESC supervisor shall notify Site Development Associates of any problems with the proposed erosion control elements, or if any revisions to the plan need to be made. Additional erosion control materials, such as filter fabric fencing, cover plastic, and straw bales, shall be kept on-site at all times in the event that an erosion control feature needs to be replaced or installed.

APPENDIX 3-A

PROJECT SWPPP

CSWPPP ANALYSIS & DESIGN

This section of the report, along with the Temporary Erosion and Sediment Control (TESC) Plan included in the engineering drawings, is intended to serve as the construction Stormwater Pollution Prevention Plan (SWPPP) for the project. The SWPPP is outlined in conformance with the 2005 edition of the Washington State Department of Ecology's Stormwater Management Manual for Western Washington (DOE Manual).

STEPS 1&2 – DATA COLLECTION & ANALYSIS

The topography of the site has been described previously in this report as being moderately sloping. The topography of the site is shown in the engineering plan set.

Soils on the project site have been identified previously in this report as being moderate to dense till, which can generally be expected to have moderate to high runoff rates with little capacity for infiltration. The existing ground cover at the project site consists mainly of forested area near the northern boundary, and pasture grass on the remainder of the site.

STEP 3 – CONSTRUCTION SWPPP DEVELOPMENT AND IMPLEMENTATION

The development and implementation of this SWPPP shall consist of 12 specific elements, as outlined in the DOE Manual. They are:

1. ***Mark Clearing Limits***

   Clearing limits will be flagged or fenced by the contractor or project surveyor prior to commencement of construction activity.

2. ***Establish Construction Access***

   A stabilized rock construction entrance will be installed at the entrance to the plat at the onset of construction.

3. ***Detain Flows***

   Prior to significant clearing, the permanent detention facility shall be constructed, so that it can be used for temporary sediment control. A temporary sediment riser shall be installed in order to ensure proper sediment control. Once the facility is constructed, the site shall be cleared and graded, and all surface water controls shall direct runoff to this facility. When final grading is complete and the site is stabilized, the temporary sediment riser shall be replaced with a permanent flow control structure.

4. ***Install Sediment Controls***

Filter fabric fencing (silt fence) shall be installed around the downstream perimeter of the site in order to keep sediment-laden stormwater from leaving the site. The fencing shall be inspected periodically to ensure its continued effectiveness.

5. ***Stabilize Soils***

The temporary erosion and sediment control plan calls for the stabilization of exposed soils through mulching or hydroseeding when the soils are not to be worked for a significant period of time. The plan also calls for the establishment of permanent vegetation through hydroseeding once the site has reached final grade.

6. ***Protect Slopes***

The northern edge of the project site shall be seeded and stabilized immediately upon reaching finished grade. Any proposed stepped lots shall also be stabilized immediately to prevent sloughing or erosion of the step slope. Any proposed rockeries or mechanically stabilized earthen walls shall have facing stones or blocks installed simultaneous to the construction of the earthen face, to provide erosion protection to the wall face.

7. ***Protect Drain Inlets***

The temporary erosion and sediment control plan calls for a filter fabric sock to be installed at all nearby catch basin inlets. Filter fabric protection shall be placed in all new catch basins as they are installed.

8. ***Stabilize Channels and Outlets***

All temporary interceptor swales shall contain check dams whenever a drop of 2 vertical feet occurs. Water discharged from the sedimentation facility shall outfall onto a rip-rap splash pad.

9. ***Control Pollutants***

All waste materials shall be disposed of in an approved location, in accordance with City of Monroe standards. In order to reasonably prevent a contamination event (such as a fuel spill), all major vehicle maintenance shall occur off-site to the greatest extent practicable. The contractor shall provide a vehicle staging area near the entrance to the site where all fueling and maintenance activity is likely to take place. This is intended to contain the area in which a contamination event is likely to take place. The contractor shall immediately contain and clean-up an area in which a contamination event occurs.

10. ***Control Dewatering***

No significant dewatering is expected to occur during this project.

11. *Maintain BMPs*

All BMPs should be monitored and maintained regularly to ensure adequate operation. A TESC supervisor shall be identified at the beginning of the project to provide monitoring and direct the appropriate maintenance activity. As site conditions change, all BMPs shall be updated as necessary to maintain compliance with City standards.

12. *Manage the Project*

The project will begin with a pre-construction conference in which an on-site TESC supervisor shall be identified. The on-site supervisor shall monitor all TESC facilities regularly and maintain a log of inspections and improvements to demonstrate compliance with City standards. The project is not large enough to be effectively phased, therefore, it will be important that the entire site is in conformance with City of Monroe erosion control standards at all times. The TESC supervisor shall notify Site Development Associates of any problems with the proposed erosion control elements, or if any revisions to the plan need to be made. Additional erosion control materials, such as filter fabric fencing, cover plastic, and straw bales, shall be kept on-site at all times in the event that an erosion control feature needs to be replaced or installed.

SECTION 4

MINIMUM REQUIREMENT #3

SOURCE CONTROL OF POLLUTION

Source Control Narrative

There are no hazardous materials proposed to be on site that would require source control BMP's.

SECTION 5

MINIMUM REQUIREMENT #4

PRESERVATION OF NATURAL DRAINAGE SYSTEMS AND OUTFALLS

NATURAL DRAINAGE COURSE DESCRIPTION

The site is currently mostly forested and wooded with some pasturelands in the southern portions of the site. There is a utility easement in the middle of the site on a N-S bearing that has a gravel maintenance road within it. This gravel road connects to chain lake road to the north. The site has two drainage basins, one that drains to the south toward the Sinclair Heights project, and one to the north that drains overland to the north, toward Chain Lake Road.

South Basin:

The south basin that contains the vast majority of the site, will contain a large detention pond. This pond will be located at the south end of the site and will be made completely of earthen berms and cut slopes. The pond will have 1' of dead storage for sediment removal and a biofiltration swale downstream of the detention pond. The biofiltration swale will discharge to a level spreader which will disperse flows into the adjacent wetland to the south of the site. The pond will be fitted with an emergency overflow structure, or "Bird Cage" that will be fitted on the frop T orifice release structure, and then a secondary emergency overflow spillway over the south bank of the detention pond. This secondary emergency overflow will be armored with quarry spalls and will also drain south into the adjacent wetland. The detention pond has been designed utilizing the latest version of WWHM3 continuous storm modeling software as per the 2005 DOE manual for existing versus proposed drainage release rates. The point of compliance is the location where the flows leave the proposed level spreader, which is the southernmost portion, and the point of the lowest elevation of the site.

North Basin:

The northern basin which is a very small portion of the site (3.6 acres of the total 35 acres), will be released to the north in its natural drainage course toward Chain Lake Road. Of the developed portion of the north basin, only the downhill 0.83 acres will be released to the north. The remainder of the plat in the north basin (2.77 acres) will be diverted to the south basin and into the proposed detention pond. This is due to the fact that the several utility (natural gas, domestic water) easements within this north basin make it very difficult to design a detention system within this north basin. And by over detaining in the south basin, within the existing detention pond, we are able to eliminate the need for two detention systems. Thus providing a more cost efficient storm drainage system, with much less maintenance for the city of Monroe and the homeowners association to operate and maintain. This 0.83 acres was chosen to keep the developed release rates .vs. pre-developed rates to the north basin within the guidelines of the 2005 DOE manual, thus meeting all Point of Compliance (POC) release rate criteria for the entire site, while utilizing one detention pond.

SECTION 6

MINIMUM REQUIREMENT #5

ON-SITE STORMWATER MANAGEMENT

Existing Site Hydrology

The site has two drainage basins, one that drains to the south toward SR 2 thru Sinclair Heights project, which we are calling the south basin. To the north is the smaller of the two basins that drains overland to the north to Chain Lake Road, and eventually west under SR 2 to the Snohomish River Valley.

South Basin:
The south basin storm runoff currently flows overland to the south and east where it enters an adjacent wetland, and then into the drainage infrastructure of the "Sinclair Heights" subdivision. This happens initially thru a drainage pipe that drains the wetland under 199th avenue SE. flows then continue to the south and then along the west side of chain lake road where they eventually go overland in a long flat wetland to the west, just after the recently proposed carriage place subdivision.

North Basin:
The North basin is the portion of the site that drains north to Chain Lake Road. Most of this drainage is in the form of sheet flow from the adjacent sites. There is a portion of this drainage that is generated from the existing utility easement maintenance road. This road drains to a roadside ditch along its west where it flows into a pipe that daylights in the Chain Lake Road roadside ditch.

Developed Site Hydrology

South Basin:

The south basin that contains the vast majority of the site, will contain a large detention pond. This pond will be located at the south end of the site and will be made completely of earthen berms and cut slopes. The pond will have 1' of dead storage for sediment removal and a biofiltration swale downstream of the detention pond. The biofiltration swale will discharge to a level spreader which will disperse flows into the adjacent wetland to the south of the site. The pond will be fitted with an emergency overflow structure, or "Bird Cage" that will be fitted on the frop T orifice release structure, and then a secondary emergency overflow spillway over the south bank of the detention pond. This secondary emergency overflow will be armored with quarry spalls and will also drain south into the adjacent wetland. The detention pond has been designed utilizing the latest version of WWHM3 continuous storm modeling software as per the 2005 DOE manual for existing versus proposed drainage release rates. The point of compliance is the location where the flows leave the proposed level spreader, which is the southernmost portion, and the point of the lowest elevation of the site.

North Basin:

The northern basin which is a very small portion of the site (3.6 acres of the total 35 acres), will be released to the north in its natural drainage course toward Chain Lake Road. Of the developed portion of the north basin, only the downhill 0.83 acres will be released to the north. The remainder of the plat in the north basin (2.77 acres) will be diverted to the south basin and into the proposed detention pond. This is due to the fact that the several utility (natural gas, domestic water) easements within this north basin make it very difficult to design a detention system within this north basin. And by over detaining in the south basin, within the existing detention pond, we are able to eliminate the need for two detention systems. Thus providing a more cost efficient storm drainage system, with much less maintenance for the city of Monroe and the homeowners association to operate and maintain. This 0.83 acres was chosen to keep the developed release rates .vs. pre-developed rates to the north basin within the guidelines of the 2005 DOE manual, thus meeting all Point of Compliance (POC) release rate criteria for the entire site, while utilizing one detention pond.

Performance Standards

Flow Control and Stormwater Quality elements are subject to the requirements of the 2005 Washington State Department of Ecology Stormwater Management Manual for the Puget Sound Basin. The modeling software used for both water quality treatment design flows and volumes, and detention volume calculations and release rates is version 3.0 of the WWHM3 Continuous Stormwater Modeling Software.

The specific performance standards the WWHM3 model uses to establish a “passing” detention facility are as follows:
Stormwater discharges shall match developed discharge durations to pre-developed durations for the range of pre-developed discharge rates from 50% of the 2-year peak flow up to the full 50-year peak flow. The pre-developed condition to be matched shall be a forested land cover unless specific basin characteristics existed otherwise prior to 1985.

SECTION 7

MINIMUM REQUIREMENT #6

RUNOFF TREATMENT

PROJECT RUNOFF TREATMENT DESIGN OVERVIEW

South Basin:
The south basin will utilize 1' of dead storage for sediment control, and a biofiltration swale designed to treat flows after detention. See calculations below from wwhm3 for water quality flow rates and for the design of the biofiltration swale as both a treatment component, and analyzed for stability in larger flows.

North Basin:

The North storm drainage basin will utilize the proposed roadside ditch in the easement connection road as a biofiltration swale to treat the undetained flows from the north basin. These flows will also be presented from wwhm3 for water quality flow rates and for the design of the biofiltration swale as both a treatment component, and analyzed for stability in larger flows

APPENDIX 7-A

WATER QUALITY
DESIGN CALCULATIONS

South Basin Biofiltration Swale Design:

From WWHM3 analysis. The Treatment flow rate is the full two year q when swale is downstream of detention, thus shown on the mitigated 2 year, the water quality section is shown below for redundancy.

Water Quality BMP Flow and Volume for POC 1.
On-line facility volume: 1.0322 acre-feet
On-line facility target flow: 0.01 cfs.
Adjusted for 15 min: 0.5502 cfs.
Off-line facility target flow: 0.3492 cfs.
Adjusted for 15 min: 0.3683 cfs.

ANALYSIS RESULTS

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	Flow(cfs)
2 year	1.14887
5 year	1.723939
10 year	2.179317
25 year	2.847225
50 year	3.416976
100 year	4.052958

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	Flow(cfs)	
2 year	0.661346	←------Treatment Flow Rate (after detention, full 2 yr.)
5 year	0.960067	
10 year	1.200398	
25 year	1.557991	
50 year	1.867299	
100 year	2.216741	←------------ Stability Flow Rate

Eaglemont (South Basin)
Biofiltration Swale Design Calculations
(Per Appendix AIII-6.1, 2005 DOE Manual)

Design Steps:

Step D-1: **Establish the design flow depth**

Design Flow Depth (y) = 4 in. <--- (Note: The swale is not to be frequently mowed, and should retain a length of 6" or more. Step D-1 calls for the design flow depth to be 2" below the winter vegetation height. Assume 4")

Step D-2: **Select the appropriate Manning's coefficient**

Manning's Coefficient (n) = 0.07 <--- (from Table 8-2.8, Chapter 8-2)

Step D-3: **Select Channel Geometry**

Swale Shape = Trapezoidal

Side Slopes = 3 :1

Channel Slope = 2 %

Step D-4: **Calculate the bottom width required to treat the 6-mo/24-hr storm event**

6-mo/24-hr Design Flowrate = 0.66 cfs <--- (2-yr release rate from the detention facility used in lieu of 6-mo. Event)

Bottom Width = 5.00 ft

Calculated Flowrate = 2.59 cfs <--- (this is the treatment capacity of the swale, and must be larger than the 6-mo/24-hr design flowrate)

Shape	y	A	P	R
Rectangular	0.3333	1.6667	5.6667	0.2941
Trapezoidal	0.3333	2.0000	7.1082	0.2814
Triangular	0.3333	0.3333	2.1082	0.1581

Step D-5: **Compute the cross sectional flow area at the calculated flowrate**

A = 2.00 ft^2

Step D-6: **Compute the flow velocity at the Design Flowrate**

V = 0.33 ft/s <-- (this velocity must be less than 1.5 ft/s to allow particle sedimentation)

Step D-7 Through Step D-15 The 1992 DOE Manual provides an approximate calculation method for the bottom width in Step D-4, to assist in hand-calculation. Steps D-7 through D-15 are intended to refine that calculation. A more accurate, iterative method was used in the calculations above, and therefore, Steps D-7 through D-15 are not necessary.

Stability Check Steps:

Step SC-1: **Calculate the 100-yr/24-hr design storm flowrate**

100-yr/24-hr Design Flowrate = 2.22 cfs <--- (see appendix 3-A)

Note: Steps SC-2, SC-3, and SC-6 through SC-9 contain an approximate method for hand-calculating the conveyance velocity during the 100-yr/24-hr event. This analysis will provide a more accurate, computer calculation, and will skip the above-listed steps:

Step SC-4: **Establish the maximum permissible velocity for erosion prevention from the following table.**

Cover	Slope %	Max. Setting Velocity (ft/s)
Kentucky Bluegrass Tall Fescue	0-5	5
Kentucky Bluegrass Tall Fescue Western Wheatgrass	5-10	4
Grass-legume Mixture	0-5 5-10	4 3
Red Fescue Redtop	0-5 5-10	2.5 Not Recommended

Selected Maximum Velocity = 4 ft/s

Step SC-5: **Select a Manning's 'n' for conveyance flows**

Manning's Coefficient (n) = 0.04

Step SC-10: **Compute the actual flow velocity for the 100-yr/24-hr storm event**

Conveyance Flow Depth (y) = 0.23 ft <-- (solved iteratively)

Channel Shape = Trapezoidal <-- (from previous page)

Bottom Width (b) = 5.00 ft <-- (from previous page)

Side Slopes (z) = 3 :1 <-- (from previous page)

Channel Slope (S) = 2.00 % <-- (from previous page)

Cross-sectional flow area (A) = 1.26 ft^2 <-- (calculated from channel geometry)

Calculated flow rate (Q_{CALC}) = 2.22 cfs <-- (this is the flow rate calculated from the conveyance flow depth above, and is provided for comparison with the 100-yr/24-hr Design Flowrate

100-yr/24-hr Design Flowrate = 2.22 cfs <-- (from above)

100-yr/24-hr Design Velocity = 1.77 fps <-- (must be less than the maximum specified in step SC-4)

<-- Use the solver to determine the flow depth
Target Cell is M105
Set target to Value of 0
By Changing Cell F95

Shape	A	P	R	Q
Rectangular	1.1500	5.4600	0.2106	2.1446
Trapezoidal	1.2558	6.4546	0.1946	2.2212
Triangular	0.1587	1.4546	0.1091	0.1909

$Q_{CALC} - Q_{100}$ = 0.00 **<-- Used in Solving for the Conveyance Flow Depth**

Final Bioswale Sizing:

Based on the previous calculations, the bioswale will require the following dimensions:

Channel Shape = Trapezoidal <-- (from page 1)

Channel Slope = 2 % <-- (from page 1)

Channel Side Slopes = 3 :1 <-- (from page 1)

100-yr/24-hr conveyance flow depth = 0.23 ft. <-- (from page 2)

Required Freeboard = 1.00 ft.

Design Swale Depth = 2.00 ft. <-- (conveyance depth + freeboard rounded up to nearest 1/2 ft.)

The 1992 DOE Manual calls for a minimum swale length of 200 ft, however, the manual allows the reduction of this length if the swale is widened to provide the same cross-sectional volume. The following calculation will determine the design width & length of the bioswale.

Required cross-sectional area (treatment) = 2.00 ft^2 <-- (from page 1)

Required treatment volume = 400.00 ft^3 <-- (treatment area * 200')

Desired Swale Length = 165 ft

Required cross-sectional treatment area = 2.42 ft^2 <-- (treatment volume / desired length)

Adjusted Bottom Width = 5 ft <-- (calculated from channel geometry maintaining the previous treatment depth)

Shape	b
Rectangular	7.2727
Trapezoidal	6.2727
Triangular	0.0000

Design Bottom Width = 5 ft <-- (adjusted bottom width, rounded up to nearest 1/2 ft)

Calculated cross-sectional treatment area = 2.00 ft^2

Shape	A	P	R	Q
Rectangular	1.6667	5.6667	0.2941	3.8830
Trapezoidal	2.0000	7.1082	0.2814	16.0786
Triangular	0.3333	2.1082	0.1581	3.2474

North Basin Biofiltration Swale Design:

The North storm drainage basin will utilize the proposed roadside ditch in the easement connection road as a biofiltration swale to treat the undetained flows from the north basin. These flows will also be presented from wwhm3 for water quality flow rates and for the design of the biofiltration swale as both a treatment component, and analyzed for stability in larger flows.

From the water quality flow page of the WWHM3 printout:

Water Quality BMP Flow and Volume for POC 1.
On-line facility volume: 0.1435 acre-feet
On-line facility target flow: 0.01 **cfs.** ←-------------------- Treatment flow rate
Adjusted for 15 min: 0.0762 **cfs.**
Off-line facility target flow: 0.0473 **cfs.**
Adjusted for 15 min: 0.0496 **cfs.**

For Stability Calculations for bioswale
MITIGATED LAND USE

ANALYSIS RESULTS

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	Flow(cfs)
2 year	0.217557
5 year	0.325868
10 year	0.411517
25 year	0.537
50 year	0.643936
100 year	0.763209

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	Flow(cfs)	
2 year	0.295045	
5 year	0.41956	
10 year	0.510242	
25 year	0.634313	
50 year	0.733733	
100 year	0.83924	←-------------------Stability Flow Rate

Biofiltration Swale Calculations: (Roadside Ditch w/Checkdamns)

Eaglemont (North Basin)
Biofiltration Swale Design Calculations
(Per Appendix AIII-6.1, 2005 DOE Manual)

Design Steps:

Step D-1: Establish the design flow depth

Design Flow Depth (y) = 4 in. <--- (Note: The swale is not to be frequently mowed, and should retain a length of 6" or more. Step D-1 calls for the design flow depth to be 2" below the winter vegetation height. Assume 4")

Step D-2: Select the appropriate Manning's coefficient

Manning's Coefficient (n) = 0.07 <— (from Table 8-2.8, Chapter 8-2)

Step D-3: Select Channel Geometry

Swale Shape = Trapezoidal

Side Slopes = 3 :1

Channel Slope = 5 %

Step D-4: Calculate the bottom width required to treat the 6-mo/24-hr storm event

6-mo/24-hr Design Flow rate = 0.07 cfs <--- (2-yr release rate from the detention facility used in lieu of 6-mo. Event)

Bottom Width = 2.00 ft

Calculated Flow rate = 1.86 cfs <--- (this is the treatment capacity of the swale, and must be larger than the 6-mo/24-hr design flow rate)

Shape	y	A	P	R
Rectangular	0.3333	0.6667	2.6667	0.2500
Trapezoidal	0.3333	1.0000	4.1082	0.2434
Triangular	0.3333	0.3333	2.1082	0.1581

Step D-5: Compute the cross sectional flow area at the calculated flow rate

A = 1.00 ft^2

Step D-6: Compute the flow velocity at the Design Flowrate

V = 0.07 ft/s <-- (this velocity must be less than 1.5 ft/s to allow particle sedimentation)

Step D-7 Through Step D-15 The 1992 DOE Manual provides an approximate calculation method for the bottom width in Step D-4, to assist in hand-calculation. Steps D-7 through D-15 are intended to refine that calculation. A more accurate, iterative method was used in the calculations above, and therefore, Steps D-7 through D-15 are not necessary.

Stability Check Steps:

Step SC-1: Calculate the 100-yr/24-hr design storm flowrate

100-yr/24-hr Design Flowrate = 0.84 cfs <-- (see appendix 3-A)

Note: Steps SC-2, SC-3, and SC-6 through SC-9 contain an approximate method for hand-calculating the conveyance velocity during the 100-yr/24-hr event. This analysis will provide a more accurate, computer calculation, and will skip the above-listed steps:

Step SC-4: Establish the maximum permissible velocity for erosion prevention from the following table.

Cover	Slope %	Max. Selling Velocity (ft/s)
Kentucky Bluegrass Tall Fescue	0-5	5
Kentucky Bluegrass Tall Fescue Western Wheatgrass	5-10	4
Grass-legume Mixture	0-5 5-10	4 3
Red Fescue Redtop	0-5 5-10	2.5 Not Recommended

Selected Maximum Velocity = 4 ft/s

Step SC-5: Select a Manning's 'n' for conveyance flows

Manning's Coefficient (n) = 0.04

Step SC-10: Compute the actual flow velocity for the 100-yr/24-hr storm event

Conveyance Flow Depth (y) = 0.18 ft <-- (solved iteratively)

Channel Shape = Trapezoidal <-- (from previous page)

Bottom Width (b) = 2.00 ft <-- (from previous page)

Side Slopes (z) = 3 :1 <-- (from previous page)

Channel Slope (S) = 5.00 % <-- (from previous page)

Cross-sectional flow area (A) = 0.41 ft^2 <-- (calculated from channel geometry)

Calculated flow rate (Q_{CALC}) = 0.89 cfs <-- (this is the flow rate calculated from the conveyance flow depth above, and is provided for comparison with the 100-yr/24-hr Design Flow rate

100-yr/24-hr Design Flowrate = 0.84 cfs <-- (from above)

100-yr/24-hr Design Velocity = 2.04 fps <-- (must be less than the maximum specified in step SC-4)

<-- Use the solver to determine the flow depth
Target Cell is M105
Set target to Value of 0
By Changing Cell F95

Shape	A	P	R	Q
Rectangular	0.3500	2.3500	0.1489	0.8191
Trapezoidal	0.4113	3.1068	0.1324	0.8897
Triangular	0.0919	1.1068	0.0830	0.1456

$Q_{CALC} - Q_{100}$ = 0.05 **<-- Used in Solving for the Conveyance Flow Depth**

Final Bioswale Sizing:

Based on the previous calculations, the bioswale will require the following dimensions:

Channel Shape = Trapezoidal <-- (from page 1)

Channel Slope = 5 % <-- (from page 1)

Channel Side Slopes = 3 :1 <-- (from page 1)

100-yr/24-hr conveyance flow depth = 0.18 ft. <-- (from page 2)

Required Freeboard = 1.00 ft.

Design Swale Depth = 2.00 ft. <-- (conveyance depth + freeboard rounded up to nearest 1/2 ft.)

The 1992 DOE Manual calls for a minimum swale length of 200 ft, however, the manual allows the reduction of this length if the swale is widened to provide the same cross-sectional volume. The following calculation will determine the design width & length of the bioswale.

Required cross-sectional area (treatment) = 1.00 ft^2 <-- (from page 1)

Required treatment volume = 200.00 ft^3 <-- (treatment area * 200')

Desired Swale Length = 165 ft

Required cross-sectional treatment area = 1.21 ft^2 <-- (treatment volume / desired length)

Adjusted Bottom Width = 5 ft <-- (calculated from channel geometry maintaining the previous treatment depth)

Shape	b
Rectangular	3.6364
Trapezoidal	2.6364
Triangular	0.0000

Design Bottom Width = 5 ft <-- (adjusted bottom width, rounded up to nearest 1/2 ft)

Calculated cross-sectional treatment area = 2.00 ft^2

Shape	A	P	R	Q
Rectangular	1.6667	5.6667	0.2941	6.1396
Trapezoidal	2.0000	7.1082	0.2814	25.4225
Triangular	0.3333	2.1082	0.1581	5.1345

Performance Standards For Water Quality Treatment:

Treatment Facility Sizing:

Water Quality Design Storm Volume: The volume of runoff predicted from a 24-hour storm with a 6-month return frequency (a.k.a., 6- month, 24-hour storm). Wetpool facilities are sized based upon the volume of runoff predicted through use of the Natural Resource Conservation Service curve number equations in Chapter 2 of Volume III, for the 6-month, 24-hour storm. Alternatively, the 91st percentile, 24-hour runoff volume indicated by an approved continuous runoff model may be used.

Water Quality Design Flow Rate:

- Preceding Detention Facilities or when Detention Facilities are not required: The flow rate at or below which 91% of the runoff volume, as estimated by an approved continuous runoff model, will be treated. Design criteria for treatment facilities are assigned to achieve the applicable performance goal at the water quality design flow rate (e.g., 80% TSS removal).

- Downstream of Detention Facilities: The full 2-year release rate from the detention facility.

Alternative methods can be used if they identify volumes and flow rates that are at least equivalent.

SECTION 8

MINIMUM REQUIREMENT #7

FLOW CONTROL

Existing Site Hydrology

The site has two drainage basins, one that drains to the south toward SR 2 thru Sinclair Heights project, which we are calling the south basin. To the north is the smaller of the two basins that drains overland to the north to Chain Lake Road, and eventually west under SR 2 to the Snohomish River Valley.

South Basin:
The south basin storm runoff currently flows overland to the south and east where it enters an adjacent wetland, and then into the drainage infrastructure of the "Sinclair Heights" subdivision. This happens initially thru a drainage pipe that drains the wetland under 199th avenue SE. flows then continue to the south and then along the west side of chain lake road where they eventually go overland in a long flat wetland to the west, just after the recently proposed carriage place subdivision.

North Basin:
The North basin is the portion of the site that drains north to Chain Lake Road. Most of this drainage is in the form of sheet flow from the adjacent sites. There is a portion of this drainage that is generated from the existing utility easement maintenance road. This road drains to a roadside ditch along its west where it flows into a pipe that daylights in the Chain Lake Road roadside ditch.

Developed Site Hydrology

South Basin:

The south basin that contains the vast majority of the site, will contain a large detention pond. This pond will be located at the south end of the site and will be made completely of earthen berms and cut slopes. The pond will have 1' of dead storage for sediment removal and a biofiltration swale downstream of the detention pond. The biofiltration swale will discharge to a level spreader which will disperse flows into the adjacent wetland to the south of the site. The pond will be fitted with an emergency overflow structure, or "Bird Cage" that will be fitted on the frop T orifice release structure, and then a secondary emergency overflow spillway over the south bank of the detention pond. This secondary emergency overflow will be armored with quarry spalls and will also drain south into the adjacent wetland. The detention pond has been designed utilizing the latest version of WWHM3 continuous storm modeling software as per the 2005 DOE manual for existing versus proposed drainage release rates. The point of compliance is the location where the flows leave the proposed level spreader, which is the southernmost portion, and the point of the lowest elevation of the site.

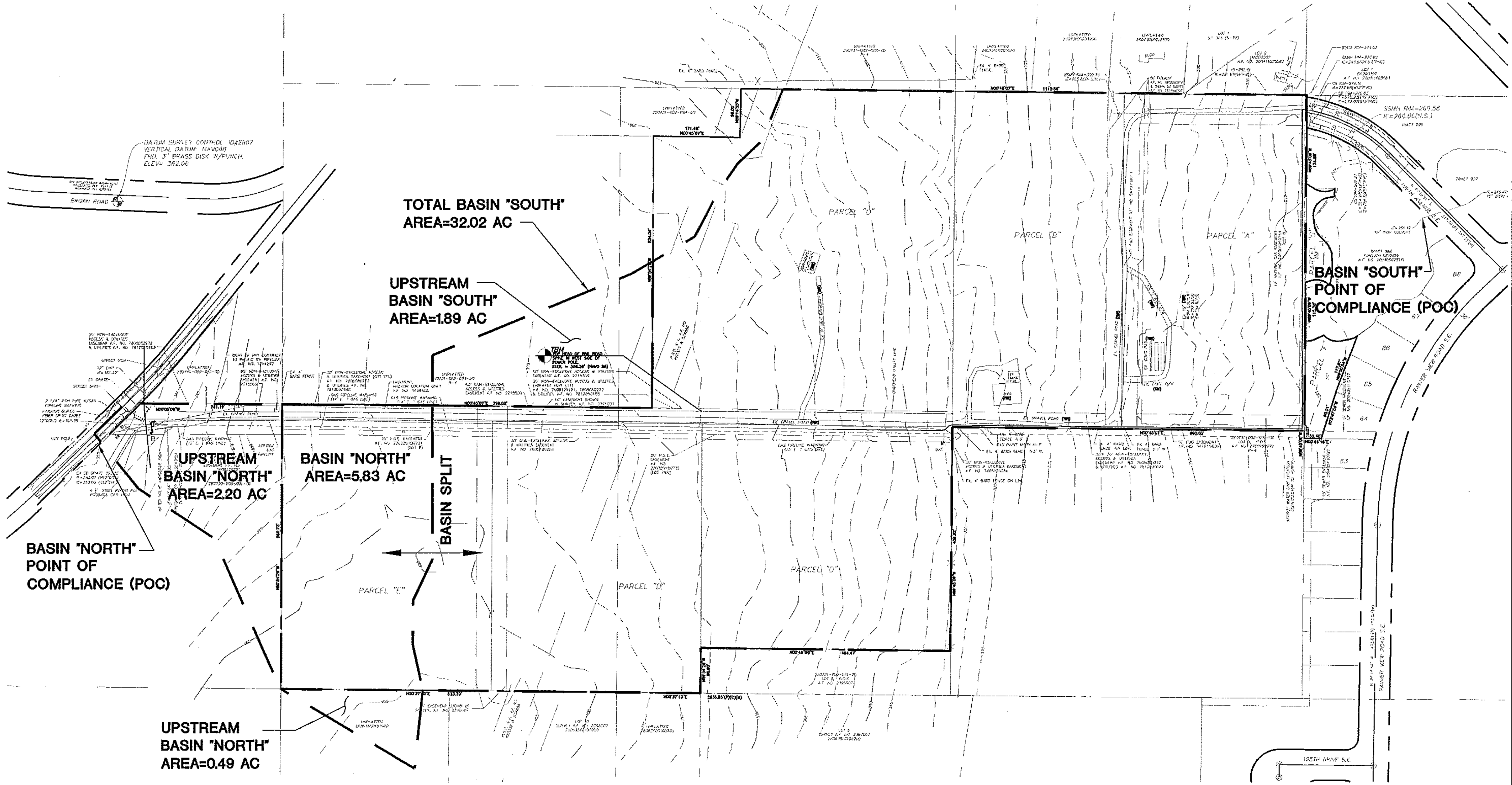
North Basin:

The northern basin which is a very small portion of the site (3.6 acres of the total 35 acres), will be released to the north in its natural drainage course toward Chain Lake Road. Of the developed portion of the north basin, only the downhill 0.83 acres will be released to the north. The remainder of the plat in the north basin (2.77 acres) will be diverted to the south basin and into the proposed detention pond. This is due to the fact that the several utility (natural gas, domestic water) easements within this north basin make it very difficult to design a detention system within this north basin. And by over detaining in the south basin, within the existing detention pond, we are able to eliminate the need for two detention systems. Thus providing a more cost efficient storm drainage system, with much less maintenance for the city of Monroe and the homeowners association to operate and maintain. This 0.83 acres was chosen to keep the developed release rates .vs. pre-developed rates to the north basin within the guidelines of the 2005 DOE manual, thus meeting all Point of Compliance (POC) release rate criteria for the entire site, while utilizing one detention pond.

Performance Standards

Flow Control and Stormwater Quality elements are subject to the requirements of the 2005 Washington State Department of Ecology Stormwater Management Manual for the Puget Sound Basin. The modeling software used for both water quality treatment design flows and volumes, and detention volume calculations and release rates is version 3.0 of the WWHM3 Continuous Stormwater Modeling Software.

The specific performance standards the WWHM3 model uses to establish a “passing” detention facility are as follows:
Stormwater discharges shall match developed discharge durations to pre-developed durations for the range of pre-developed discharge rates from 50% of the 2-year peak flow up to the full 50-year peak flow. The pre-developed condition to be matched shall be a forested land cover unless specific basin characteristics existed otherwise prior to 1985.



1" = 200' 0 200

ACR Design SJR Drawn 10/10/2012 Date 278-003-12 Project No.	SDA Civil Engineering, Project Management, Planning 1724 W. Marine View Drive, Suite 140; Everett, Wasington 98201 Office: 425.486.6533 Fax: 425.486.6593 www.sdaengineers.com	**EAGLEMONT** EXISTING DRAINAGE BASIN MAP Sheet: 1

Oct 10, 2012 - 7:23PM Last Saved By: SRose

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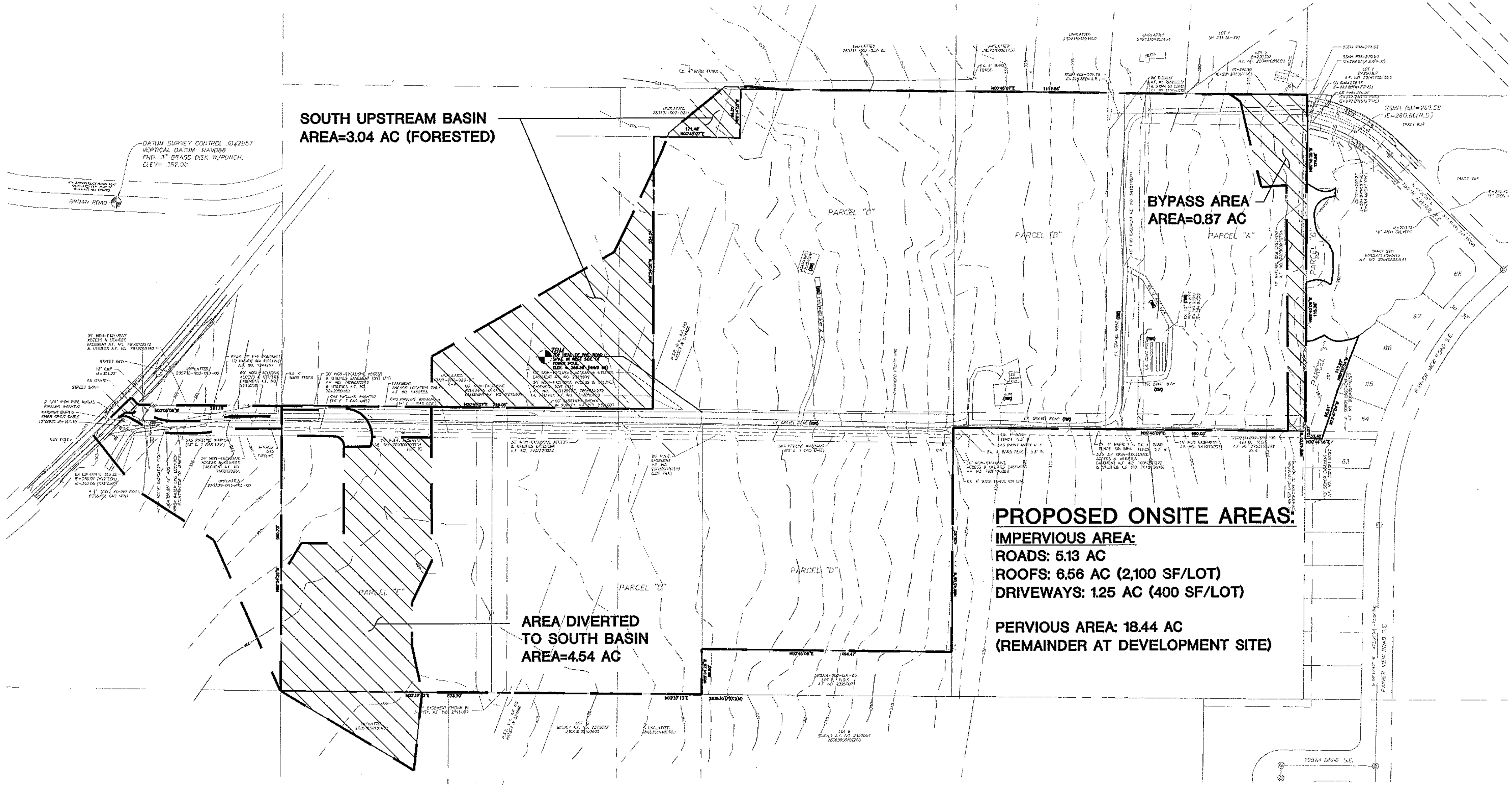

ACR
Design
SJR
Drawn
10/10/2012
Date
278-003-12
Project No.

SDA
Civil Engineering
Project Management
Planning
1724 W. Marine View Drive, Suite 140; Everett, Wasington 98201
Office: 425.486.6533 Fax: 425.486.6593 www.sdaengineers.com

EAGLEMONT

PROPOSED SOUTH
DRAINAGE BASIN MAP

Sheet: 1

Oct 11, 2012 - 8:17AM Last Saved By: SRose

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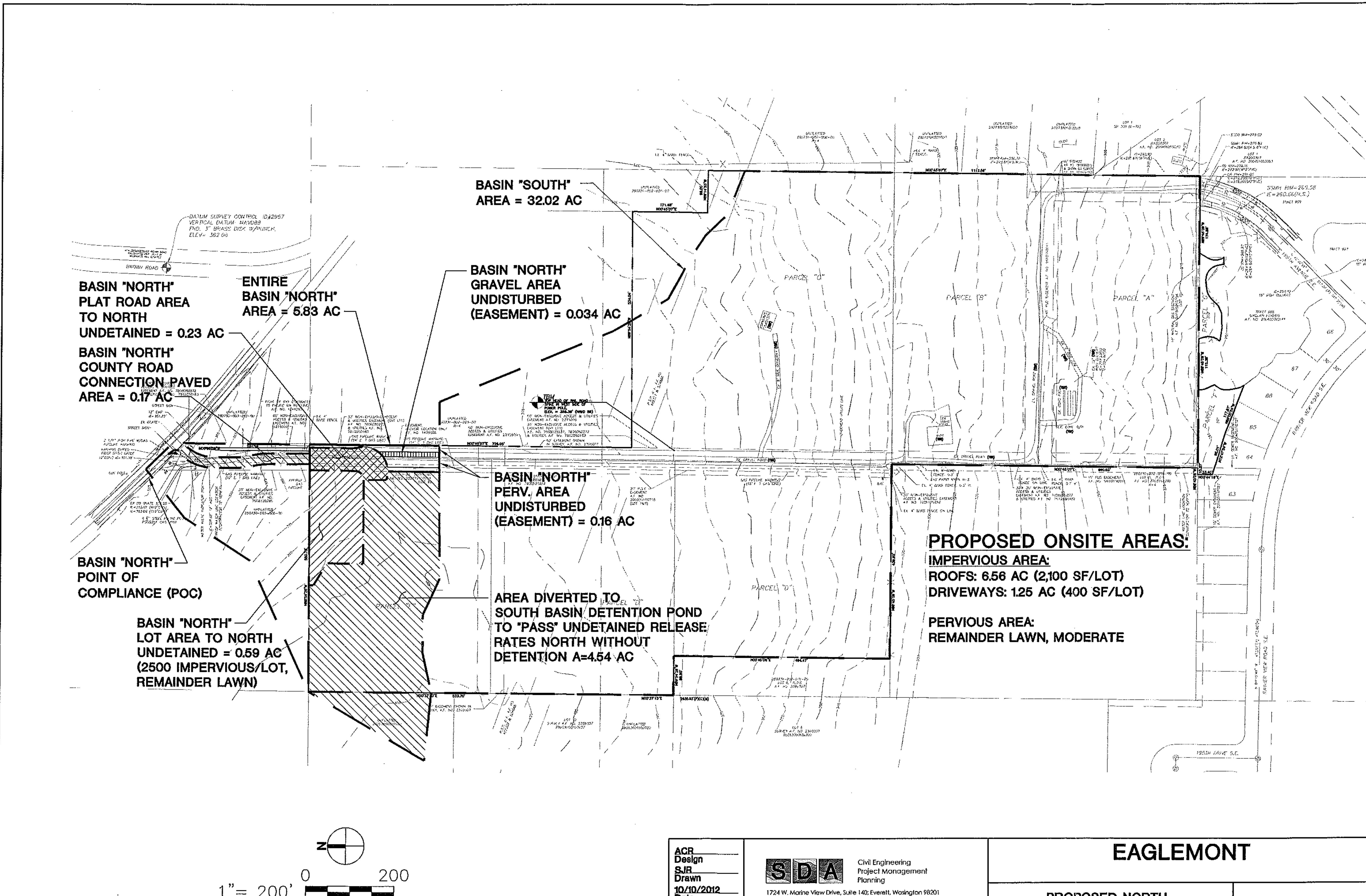

ACR Design SJR Drawn 10/10/2012 Date 278-003-12 Project No.	SDA Civil Engineering Project Management Planning 1724 W. Marine View Drive, Suite 140; Everett, Wasington 98201 Office: 425.486.6533 Fax: 425.486.6593 www.sdaengineers.com	EAGLEMONT PROPOSED NORTH DRAINAGE BASIN MAP Sheet: 1

Oct 11, 2012 - 8:18AM Last Saved By: SRose

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APPENDIX 8-A

FLOW CONTROL DESIGN CALCULATIONS

South Basin Detention Calculations

Western Washington Hydrology Model
PROJECT REPORT

Project Name: Eaglemont South Basin (Main Pond)
Site Address:
City : Monroe
Report Date : 10/9/2012
Gage : Everett
Data Start : 1948/10/01
Data End : 1997/09/30
Precip Scale: 1.20
WWHM3 Version:

PREDEVELOPED LAND USE

Name : Basin 1
Bypass: No

GroundWater: No

Pervious Land Use	Acres
C, Forest, Mod	36.56

Impervious Land Use	Acres

Element Flows To:

Surface	Interflow	Groundwater

Name : Basin 1
Bypass: No

GroundWater: No

Pervious Land Use	Acres
C, Lawn, Mod	18.44
C, Forest, Mod	3.04

Impervious Land Use	Acres
ROADS MOD	5.13
ROOF TOPS FLAT	6.56
DRIVEWAYS MOD	1.25
POND	1.16

Element Flows To:

Surface	**Interflow**	**Groundwater**
Trapezoidal Pond 1,	Trapezoidal Pond 1,	

Name : Trapezoidal Pond 1
Bottom Length: 344ft.
Bottom Width: 55ft.
Depth : 15ft.
Volume at riser head : 11.5284ft.
Side slope 1: 3 To 1
Side slope 2: 3 To 1
Side slope 3: 2 To 1
Side slope 4: 3 To 1
Discharge Structure
Riser Height: 14 ft.
Riser Diameter: 18 in.
NotchType : Rectangular
Notch Width : 0.250 ft.
Notch Height: 1.500 ft.
Orifice 1 Diameter: 2.9375 in. **Elevation:** 0 ft.
Orifice 1 Diameter: 4.875 in. **Elevation:** 9 ft.
Orifice 1 Diameter: 3 in. **Elevation:** 10.25 ft.

Element Flows To:

Outlet 1	**Outlet 2**

Pond Hydraulic Table

Stage(ft)	Area(acr)	Volume(acr-ft)	Dschrg(cfs)	Infilt(cfs)
0.000	0.434	0.000	0.000	0.000
0.167	0.442	0.073	0.093	0.000
0.333	0.450	0.147	0.131	0.000
0.500	0.458	0.223	0.160	0.000
0.667	0.466	0.300	0.185	0.000
0.833	0.474	0.378	0.207	0.000
1.000	0.482	0.458	0.227	0.000
1.167	0.490	0.539	0.245	0.000
1.333	0.498	0.622	0.262	0.000
1.500	0.506	0.705	0.278	0.000
1.667	0.515	0.790	0.293	0.000
1.833	0.523	0.877	0.307	0.000
2.000	0.531	0.965	0.321	0.000
2.167	0.540	1.054	0.334	0.000
2.333	0.548	1.145	0.346	0.000
2.500	0.556	1.237	0.358	0.000
2.667	0.565	1.330	0.370	0.000
2.833	0.573	1.425	0.381	0.000
3.000	0.582	1.521	0.393	0.000
3.167	0.590	1.619	0.403	0.000
3.333	0.599	1.718	0.414	0.000
3.500	0.607	1.818	0.424	0.000
3.667	0.616	1.920	0.434	0.000
3.833	0.625	2.024	0.444	0.000

4.000	0.634	2.129	0.453	0.000
4.167	0.642	2.235	0.463	0.000
4.333	0.651	2.343	0.472	0.000
4.500	0.660	2.452	0.481	0.000
4.667	0.669	2.563	0.490	0.000
4.833	0.678	2.675	0.498	0.000
5.000	0.687	2.789	0.507	0.000
5.167	0.696	2.904	0.515	0.000
5.333	0.705	3.021	0.523	0.000
5.500	0.714	3.139	0.531	0.000
5.667	0.723	3.259	0.539	0.000
5.833	0.732	3.380	0.547	0.000
6.000	0.742	3.503	0.555	0.000
6.167	0.751	3.627	0.563	0.000
6.333	0.760	3.753	0.570	0.000
6.500	0.769	3.880	0.578	0.000
6.667	0.779	4.009	0.585	0.000
6.833	0.788	4.140	0.592	0.000
7.000	0.798	4.272	0.600	0.000
7.167	0.807	4.406	0.607	0.000
7.333	0.816	4.541	0.614	0.000
7.500	0.826	4.678	0.621	0.000
7.667	0.836	4.817	0.628	0.000
7.833	0.845	4.957	0.634	0.000
8.000	0.855	5.098	0.641	0.000
8.167	0.865	5.242	0.648	0.000
8.333	0.874	5.386	0.654	0.000
8.500	0.884	5.533	0.661	0.000
8.667	0.894	5.681	0.667	0.000
8.833	0.904	5.831	0.674	0.000
9.000	0.914	5.982	0.680	0.000
9.167	0.924	6.136	0.941	0.000
9.333	0.934	6.290	1.053	0.000
9.500	0.944	6.447	1.140	0.000
9.667	0.954	6.605	1.214	0.000
9.833	0.964	6.765	1.280	0.000
10.00	0.974	6.926	1.341	0.000
10.17	0.984	7.089	1.397	0.000
10.33	0.994	7.254	1.517	0.000
10.50	1.004	7.421	1.617	0.000
10.67	1.015	7.589	1.699	0.000
10.83	1.025	7.759	1.772	0.000
11.00	1.035	7.931	1.839	0.000
11.17	1.046	8.104	1.902	0.000
11.33	1.056	8.279	1.962	0.000
11.50	1.067	8.456	2.020	0.000
11.67	1.077	8.635	2.075	0.000
11.83	1.088	8.815	2.128	0.000
12.00	1.098	8.997	2.179	0.000
12.17	1.109	9.181	2.228	0.000
12.33	1.120	9.367	2.277	0.000
12.50	1.130	9.554	2.324	0.000
12.67	1.141	9.744	2.424	0.000
12.83	1.152	9.935	2.563	0.000
13.00	1.163	10.13	2.722	0.000
13.17	1.173	10.32	2.893	0.000
13.33	1.184	10.52	3.070	0.000

13.50	1.195	10.72	3.249	0.000
13.67	1.206	10.92	3.462	0.000
13.83	1.217	11.12	3.688	0.000
14.00	1.228	11.32	4.265	0.000
14.17	1.239	11.53	5.297	0.000
14.33	1.250	11.74	7.152	0.000
14.50	1.262	11.95	9.542	0.000
14.67	1.273	12.16	12.37	0.000
14.83	1.284	12.37	15.56	0.000
15.00	1.295	12.58	19.09	0.000
15.17	1.307	12.80	22.93	0.000

Name : South Basin (Bypass)
Bypass: Yes

GroundWater: No

Pervious Land Use	Acres
C, Lawn, Mod	.63

Impervious Land Use	Acres
ROADS MOD	0.24
ROOF TOPS FLAT	0.12

Element Flows To:

Surface	Interflow	Groundwater
Trapezoidal Pond 1,	Trapezoidal Pond 1,	

MITIGATED LAND USE

ANALYSIS RESULTS

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	Flow(cfs)
2 year	1.200078
5 year	1.800779
10 year	2.276454
25 year	2.974131
50 year	3.569277
100 year	4.233606

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	Flow(cfs)
2 year	0.649225
5 year	0.93754
10 year	1.173268
25 year	1.52875
50 year	1.840028
100 year	2.195279

Yearly Peaks for Predeveloped and Mitigated. POC #1

Year	Predeveloped	Mitigated
1950	0.800	0.537
1951	2.256	0.623
1952	0.804	0.546
1953	0.952	0.509
1954	1.281	0.499
1955	2.093	0.607
1956	1.956	1.053
1957	1.317	1.135
1958	2.113	0.931
1959	2.029	0.580
1960	1.146	0.616
1961	1.037	0.638
1962	1.433	0.669
1963	1.837	0.548
1964	2.909	0.536
1965	1.029	0.489
1966	1.005	0.635
1967	0.615	0.520
1968	1.319	0.549
1969	1.485	0.663
1970	2.250	0.563
1971	0.798	0.529
1972	1.265	1.128
1973	0.974	0.594
1974	0.825	0.598
1975	1.081	0.607
1976	0.880	0.505
1977	0.765	0.593
1978	0.703	0.518
1979	0.873	0.508
1980	3.101	0.577
1981	0.887	0.500
1982	1.110	0.523
1983	0.954	0.665
1984	1.141	0.534
1985	1.091	1.250
1986	1.529	1.087
1987	3.201	2.058
1988	1.542	1.598
1989	0.773	0.652
1990	1.501	0.464
1991	1.038	0.659
1992	1.083	0.632
1993	1.013	0.645
1994	0.616	0.472
1995	0.683	0.612
1996	1.050	1.030
1997	1.865	0.971
1998	4.314	3.304

Ranked Yearly Peaks for Predeveloped and Mitigated. POC #1

Rank	Predeveloped	Mitigated

1	4.3145	3.3045
2	3.2006	2.0581
3	3.1011	1.5985
4	2.9093	1.2498
5	2.2557	1.1349
6	2.2501	1.1281
7	2.1133	1.0873
8	2.0931	1.0530
9	2.0292	1.0296
10	1.9561	0.9715
11	1.8653	0.9306
12	1.8369	0.6686
13	1.5416	0.6650
14	1.5292	0.6634
15	1.5012	0.6589
16	1.4849	0.6518
17	1.4327	0.6448
18	1.3189	0.6378
19	1.3174	0.6351
20	1.2815	0.6315
21	1.2650	0.6227
22	1.1464	0.6161
23	1.1413	0.6117
24	1.1100	0.6071
25	1.0914	0.6066
26	1.0832	0.5977
27	1.0808	0.5944
28	1.0499	0.5926
29	1.0376	0.5799
30	1.0372	0.5766
31	1.0289	0.5626
32	1.0128	0.5487
33	1.0048	0.5485
34	0.9738	0.5459
35	0.9540	0.5369
36	0.9517	0.5363
37	0.8874	0.5335
38	0.8796	0.5292
39	0.8735	0.5230
40	0.8250	0.5195
41	0.8039	0.5179
42	0.7996	0.5091
43	0.7978	0.5075
44	0.7725	0.5045
45	0.7648	0.5005
46	0.7026	0.4993
47	0.6830	0.4892
48	0.6157	0.4721
49	0.6152	0.4641

POC #1
The Facility PASSED

The Facility PASSED.

Flow(CFS) Predev Dev Percentage Pass/Fail

0.6000	3378	2897	85	Pass
0.6300	2946	1815	61	Pass
0.6600	2579	975	37	Pass
0.6900	2250	492	21	Pass
0.7200	1943	476	24	Pass
0.7500	1701	447	26	Pass
0.7800	1481	419	28	Pass
0.8100	1278	398	31	Pass
0.8400	1121	380	33	Pass
0.8700	990	359	36	Pass
0.9000	858	343	39	Pass
0.9300	747	329	44	Pass
0.9599	640	309	48	Pass
0.9899	558	278	49	Pass
1.0199	491	260	52	Pass
1.0499	429	243	56	Pass
1.0799	385	223	57	Pass
1.1099	347	205	59	Pass
1.1399	311	187	60	Pass
1.1699	278	179	64	Pass
1.1999	251	171	68	Pass
1.2299	233	163	69	Pass
1.2599	216	153	70	Pass
1.2899	202	147	72	Pass
1.3199	185	141	76	Pass
1.3498	174	135	77	Pass
1.3798	161	130	80	Pass
1.4098	154	125	81	Pass
1.4398	142	122	85	Pass
1.4698	135	119	88	Pass
1.4998	130	115	88	Pass
1.5298	123	112	91	Pass
1.5598	119	108	90	Pass
1.5898	114	102	89	Pass
1.6198	112	99	88	Pass
1.6498	109	97	88	Pass
1.6798	103	95	92	Pass
1.7098	97	91	93	Pass
1.7397	95	89	93	Pass
1.7697	94	86	91	Pass
1.7997	91	85	93	Pass
1.8297	89	83	93	Pass
1.8597	85	79	92	Pass
1.8897	81	76	93	Pass
1.9197	76	73	96	Pass
1.9497	74	69	93	Pass
1.9797	72	62	86	Pass
2.0097	69	59	85	Pass
2.0397	67	55	82	Pass
2.0697	66	48	72	Pass
2.0997	63	45	71	Pass
2.1296	61	44	72	Pass
2.1596	60	43	71	Pass
2.1896	58	42	72	Pass
2.2196	57	40	70	Pass
2.2496	55	37	67	Pass
2.2796	52	32	61	Pass

2.3096	51	27	52	Pass
2.3396	48	25	52	Pass
2.3696	45	24	53	Pass
2.3996	44	23	52	Pass
2.4296	42	22	52	Pass
2.4596	39	22	56	Pass
2.4896	38	20	52	Pass
2.5195	36	20	55	Pass
2.5495	35	19	54	Pass
2.5795	34	18	52	Pass
2.6095	34	18	52	Pass
2.6395	32	17	53	Pass
2.6695	31	17	54	Pass
2.6995	30	15	50	Pass
2.7295	29	15	51	Pass
2.7595	28	14	50	Pass
2.7895	24	14	58	Pass
2.8195	24	13	54	Pass
2.8495	23	13	56	Pass
2.8795	20	12	60	Pass
2.9094	20	12	60	Pass
2.9394	18	11	61	Pass
2.9694	16	10	62	Pass
2.9994	16	10	62	Pass
3.0294	13	9	69	Pass
3.0594	13	9	69	Pass
3.0894	13	8	61	Pass
3.1194	11	7	63	Pass
3.1494	11	6	54	Pass
3.1794	11	5	45	Pass
3.2094	8	5	62	Pass
3.2394	7	4	57	Pass
3.2694	6	4	66	Pass
3.2993	6	1	16	Pass
3.3293	5	0	0	Pass
3.3593	3	0	0	Pass
3.3893	3	0	0	Pass
3.4193	3	0	0	Pass
3.4493	3	0	0	Pass
3.4793	2	0	0	Pass
3.5093	2	0	0	Pass
3.5393	2	0	0	Pass
3.5693	2	0	0	Pass

Water Quality BMP Flow and Volume for POC 1.
On-line facility volume: 1.0579 acre-feet
On-line facility target flow: 0.01 **cfs.**
Adjusted for 15 min: 0.5631 **cfs.**
Off-line facility target flow: 0.3536 **cfs.**
Adjusted for 15 min: 0.3718 **cfs.**

Perlnd and Implnd Changes

No changes have been made.

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North Basin Detention Calculations:

Actually, this is to show that the bypassed area has a passing flow rate for the criteris of wwhm3 for the north basin, based on the existing basin size to the north, and the developed basin size with the correct amount diverted south. Thus the north basin of the pond, plus the existing basin area upstream of the POC is within the allowable release rates of WWHM3.

```
         Western Washington Hydrology Model
                  PROJECT REPORT
___________________________________________________________________

Project Name:  Eaglemont
Site Address:  North Basin
City        :  Monroe
Report Date :  9/26/2012
Gage        :  Everett
Data Start  :  1948/10/01
Data End    :  1997/09/30
Precip Scale:  1.20
WWHM3 Version:
___________________________________________________________________

PREDEVELOPED LAND USE

Name      :  North Basin
Bypass:  No

GroundWater:  No

Pervious Land Use          Acres
 C, Forest, Mod              6.11
 C, Pasture, Mod             .17

Impervious Land Use        Acres
 DRIVEWAYS MOD               0.034

___________________________________________________________________

Element Flows To:
Surface              Interflow              Groundwater
___________________________________________________________________

Name      :  Basin  1
Bypass:  No

GroundWater:  No

Pervious Land Use          Acres
 C, Forest, Mod              1.91
 C, Pasture, Mod             .105
 C, Lawn, Mod                .32

Impervious Land Use        Acres
```

ROADS MOD	0.34
ROOF TOPS FLAT	0.22

Element Flows To:

Surface	Interflow	Groundwater

MITIGATED LAND USE

ANALYSIS RESULTS

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	Flow(cfs)
2 year	0.217557
5 year	0.325868
10 year	0.411517
25 year	0.537
50 year	0.643936
100 year	0.763209

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	Flow(cfs)
2 year	0.295045
5 year	0.41956
10 year	0.510242
25 year	0.634313
50 year	0.733733
100 year	0.83924

Yearly Peaks for Predeveloped and Mitigated. POC #1

Year	Predeveloped	Mitigated
1950	0.153	0.266
1951	0.413	0.504
1952	0.146	0.261
1953	0.177	0.252
1954	0.238	0.350
1955	0.381	0.437
1956	0.351	0.404
1957	0.234	0.194
1958	0.380	0.383
1959	0.380	0.568
1960	0.206	0.230
1961	0.183	0.266
1962	0.267	0.704
1963	0.334	0.377
1964	0.529	0.583
1965	0.185	0.217
1966	0.176	0.178
1967	0.113	0.193
1968	0.234	0.537

1969	0.263	0.322
1970	0.424	0.607
1971	0.140	0.210
1972	0.230	0.353
1973	0.202	0.561
1974	0.149	0.305
1975	0.197	0.316
1976	0.169	0.312
1977	0.136	0.228
1978	0.129	0.176
1979	0.159	0.196
1980	0.558	0.540
1981	0.158	0.198
1982	0.202	0.243
1983	0.172	0.214
1984	0.209	0.294
1985	0.192	0.267
1986	0.278	0.325
1987	0.561	0.477
1988	0.270	0.328
1989	0.141	0.248
1990	0.277	0.363
1991	0.182	0.176
1992	0.190	0.184
1993	0.186	0.251
1994	0.109	0.229
1995	0.122	0.153
1996	0.184	0.180
1997	0.332	0.294
1998	0.761	0.565

Ranked Yearly Peaks for Predeveloped and Mitigated. POC #1

Rank	Predeveloped	Mitigated
1	0.7609	0.7037
2	0.5606	0.6074
3	0.5577	0.5833
4	0.5286	0.5682
5	0.4237	0.5649
6	0.4126	0.5607
7	0.3813	0.5397
8	0.3804	0.5365
9	0.3800	0.5043
10	0.3509	0.4771
11	0.3340	0.4368
12	0.3321	0.4044
13	0.2775	0.3825
14	0.2768	0.3773
15	0.2704	0.3627
16	0.2674	0.3534
17	0.2632	0.3497
18	0.2385	0.3278
19	0.2344	0.3250
20	0.2339	0.3223
21	0.2303	0.3158
22	0.2085	0.3121
23	0.2060	0.3055

24	0.2025	0.2943
25	0.2025	0.2939
26	0.1973	0.2673
27	0.1919	0.2663
28	0.1903	0.2655
29	0.1864	0.2611
30	0.1849	0.2525
31	0.1840	0.2505
32	0.1834	0.2485
33	0.1825	0.2430
34	0.1769	0.2305
35	0.1757	0.2286
36	0.1716	0.2276
37	0.1691	0.2172
38	0.1591	0.2141
39	0.1576	0.2097
40	0.1527	0.1975
41	0.1491	0.1957
42	0.1462	0.1938
43	0.1406	0.1927
44	0.1404	0.1841
45	0.1360	0.1797
46	0.1294	0.1777
47	0.1219	0.1762
48	0.1133	0.1759
49	0.1090	0.1533

POC #1
The Facility PASSED

The Facility PASSED.

Flow(CFS)	Predev	Dev	Percentage	Pass/Fail
0.1088	3102	1484	47	Pass
0.1142	2657	1284	48	Pass
0.1196	2340	1128	48	Pass
0.1250	2038	992	48	Pass
0.1304	1735	839	48	Pass
0.1358	1521	755	49	Pass
0.1412	1281	645	50	Pass
0.1466	1125	585	52	Pass
0.1520	985	511	51	Pass
0.1574	867	461	53	Pass
0.1628	756	412	54	Pass
0.1682	641	364	56	Pass
0.1736	554	328	59	Pass
0.1791	484	280	57	Pass
0.1845	421	258	61	Pass
0.1899	380	237	62	Pass
0.1953	339	212	62	Pass
0.2007	313	194	61	Pass
0.2061	270	183	67	Pass
0.2115	246	164	66	Pass
0.2169	226	151	66	Pass
0.2223	207	131	63	Pass
0.2277	196	120	61	Pass

0.2331	181	108	59	Pass
0.2385	169	105	62	Pass
0.2439	158	102	64	Pass
0.2493	150	96	64	Pass
0.2547	137	87	63	Pass
0.2601	132	81	61	Pass
0.2655	126	78	61	Pass
0.2709	122	73	59	Pass
0.2764	119	67	56	Pass
0.2818	113	62	54	Pass
0.2872	109	60	55	Pass
0.2926	103	56	54	Pass
0.2980	98	52	53	Pass
0.3034	95	51	53	Pass
0.3088	93	49	52	Pass
0.3142	90	45	50	Pass
0.3196	88	42	47	Pass
0.3250	86	41	47	Pass
0.3304	84	39	46	Pass
0.3358	79	37	46	Pass
0.3412	74	36	48	Pass
0.3466	72	35	48	Pass
0.3520	68	29	42	Pass
0.3574	67	28	41	Pass
0.3628	66	27	40	Pass
0.3682	64	23	35	Pass
0.3737	64	22	34	Pass
0.3791	62	20	32	Pass
0.3845	57	19	33	Pass
0.3899	56	18	32	Pass
0.3953	55	17	30	Pass
0.4007	53	17	32	Pass
0.4061	52	15	28	Pass
0.4115	48	15	31	Pass
0.4169	44	15	34	Pass
0.4223	43	15	34	Pass
0.4277	40	15	37	Pass
0.4331	38	15	39	Pass
0.4385	36	13	36	Pass
0.4439	34	12	35	Pass
0.4493	34	12	35	Pass
0.4547	32	12	37	Pass
0.4601	32	12	37	Pass
0.4656	30	12	40	Pass
0.4710	29	12	41	Pass
0.4764	29	12	41	Pass
0.4818	24	10	41	Pass
0.4872	24	10	41	Pass
0.4926	23	10	43	Pass
0.4980	20	10	50	Pass
0.5034	20	10	50	Pass
0.5088	18	9	50	Pass
0.5142	18	9	50	Pass
0.5196	16	9	56	Pass
0.5250	15	9	60	Pass
0.5304	13	9	69	Pass
0.5358	12	9	75	Pass

0.5412	12	6	50	Pass
0.5466	12	6	50	Pass
0.5520	11	6	54	Pass
0.5574	11	6	54	Pass
0.5629	7	5	71	Pass
0.5683	6	4	66	Pass
0.5737	6	3	50	Pass
0.5791	5	3	60	Pass
0.5845	5	2	40	Pass
0.5899	3	2	66	Pass
0.5953	3	2	66	Pass
0.6007	3	2	66	Pass
0.6061	2	2	100	Pass
0.6115	2	1	50	Pass
0.6169	2	1	50	Pass
0.6223	2	1	50	Pass
0.6277	1	1	100	Pass
0.6331	1	1	100	Pass
0.6385	1	1	100	Pass
0.6439	1	1	100	Pass

Water Quality BMP Flow and Volume for POC 1.
On-line facility volume: 0 acre-feet
On-line facility target flow: 0 cfs.
Adjusted for 15 min: 0 cfs.
Off-line facility target flow: 0 cfs.
Adjusted for 15 min: 0 cfs.

Perlnd and Implnd Changes

No changes have been made.

SECTION 9

MINIMUM REQUIREMENT #8

WETLANDS PROTECTION

PROJECT OVERVIEW

There are no Wetland on this site.

SECTION 10

MINIMUM REQUIREMENT #9

BASIN/WATERSHED PLANNING

All Basin and Watershed Planning issues are discussed in Section #4 of this report that discussed discharging into the natural locations.

SECTION 11

MINIMUM REQUIREMENT #10

OPERATION AND MAINTENANCE

Operation and Maintenance Section:

No. 1 – Detention Ponds

Maintenance Component	Defect	Conditions When Maintenance Is Needed	Results Expected When Maintenance Is Performed
General	Trash & Debris	Any trash and debris which exceed 5 cubic feet per 1,000 square feet (this is about equal to the amount of trash it would take to fill up one standard size garbage can). In general, there should be no visual evidence of dumping. If less than threshold all trash and debris will be removed as part of next scheduled maintenance.	Trash and debris cleared from site.
	Poisonous Vegetation and noxious weeds	Any poisonous or nuisance vegetation which may constitute a hazard to maintenance personnel or the public. Any evidence of noxious weeds as defined by State or local regulations. (Apply requirements of adopted IPM policies for the use of herbicides).	No danger of poisonous vegetation where maintenance personnel or the public might normally be. (Coordinate with local health department) Complete eradication of noxious weeds may not be possible. Compliance with State or local eradication policies required
	Contaminants and Pollution	Any evidence of oil, gasoline, contaminants or other pollutants (Coordinate removal/cleanup with local water quality response agency).	~~No contaminants or pollutants present.~~
	Rodent Holes	Any evidence of rodent holes if facility is acting as a dam or berm, or any evidence of water piping through dam or berm via rodent holes.	Rodents destroyed and dam or berm repaired. (Coordinate with local health department; coordinate with Ecology Dam Safety Office if pond exceeds 10 acre-feet.)

Maintenance Component	Defect	Conditions When Maintenance Is Needed	Results Expected When Maintenance Is Performed
	Beaver Dams	Dam results in change or function of the facility.	Facility is returned to design function. (Coordinate trapping of beavers and removal of dams with appropriate permitting agencies)
	Insects	When insects such as wasps and hornets interfere with maintenance activities.	Insects destroyed or removed from site. Apply insecticides in compliance with adopted IPM policies
	Tree Growth and Hazard Trees	Tree growth does not allow maintenance access or interferes with maintenance activity (i.e., slope mowing, silt removal, vactoring, or equipment movements). If trees are not interfering with access or maintenance, do not remove If dead, diseased, or dying trees are identified (Use a certified Arborist to determine health of tree or removal requirements)	Trees do not hinder maintenance activities. Harvested trees should be recycled into mulch or other beneficial uses (e.g., alders for firewood). Remove hazard Trees
Side Slopes of Pond	Erosion	Eroded damage over 2 inches deep where cause of damage is still present or where there is potential for continued erosion. Any erosion observed on a compacted berm embankment.	Slopes should be stabilized using appropriate erosion control measure(s); e.g., rock reinforcement, planting of grass, compaction. If erosion is occurring on compacted berms a licensed civil engineer should be consulted to resolve source of erosion.
Storage Area	Sediment	Accumulated sediment that exceeds 10% of the designed pond depth unless otherwise specified or affects inletting or outletting condition of the facility.	Sediment cleaned out to designed pond shape and depth; pond reseeded if necessary to control erosion.
	Liner (If Applicable)	Liner is visible and has more than three 1/4-inch holes in it.	Liner repaired or replaced. Liner is fully covered.

Maintenance Component	Defect	Conditions When Maintenance Is Needed	Results Expected When Maintenance Is Performed
Pond Berms (Dikes)	Settlements	Any part of berm which has settled 4 inches lower than the design elevation. If settlement is apparent, measure berm to determine amount of settlement. Settling can be an indication of more severe problems with the berm or outlet works. A licensed civil engineer should be consulted to determine the source of the settlement.	Dike is built back to the design elevation.
	Piping	Discernable water flow through pond berm. Ongoing erosion with potential for erosion to continue. (Recommend a Goethechnical engineer be called in to inspect and evaluate condition and recommend repair of condition.	Piping eliminated. Erosion potential resolved.
Emergency Overflow/ Spillway and Berms over 4 feet in height.	Tree Growth	Tree growth on emergency spillways creates blockage problems and may cause failure of the berm due to uncontrolled overtopping. Tree growth on berms over 4 feet in height may lead to piping through the berm which could lead to failure of the berm.	Trees should be removed. If root system is small (base less than 4 inches) the root system may be left in place. Otherwise the roots should be removed and the berm restored. A licensed civil engineer should be consulted for proper berm/spillway restoration.
	Piping	Discernable water flow through pond berm. Ongoing erosion with potential for erosion to continue. (Recommend a Goethechnical engineer be called in to inspect and evaluate condition and recommend repair of condition.	Piping eliminated. Erosion potential resolved.
Emergency Overflow/ Spillway	Emergency Overflow/ Spillway	Only one layer of rock exists above native soil in area five square feet or larger, or any exposure of native soil at the top of out flow path of spillway. (Rip-rap on inside slopes need not be replaced.)	Rocks and pad depth are restored to design standards.
	Erosion	See "Side Slopes of Pond"	

No. 4 – Control Structure/Flow Restrictor

Maintenance Component	Defect	Condition When Maintenance is Needed	Results Expected When Maintenance is Performed
General	Trash and Debris (Includes Sediment)	Material exceeds 25% of sump depth or 1 foot below orifice plate.	Control structure orifice is not blocked. All trash and debris removed.

	Structural Damage	Structure is not securely attached to manhole wall.	Structure securely attached to wall and outlet pipe.
		Structure is not in upright position (allow up to 10% from plumb).	Structure in correct position.
		Connections to outlet pipe are not watertight and show signs of rust.	Connections to outlet pipe are water tight; structure repaired or replaced and works as designed.
		Any holes--other than designed holes--in the structure.	Structure has no holes other than designed holes.
Cleanout Gate	Damaged or Missing	Cleanout gate is not watertight or is missing.	Gate is watertight and works as designed.
		Gate cannot be moved up and down by one maintenance person.	Gate moves up and down easily and is watertight.
		Chain/rod leading to gate is missing or damaged.	Chain is in place and works as designed.
		Gate is rusted over 50% of its surface area.	Gate is repaired or replaced to meet design standards.
Orifice Plate	Damaged or Missing	Control device is not working properly due to missing, out of place, or bent orifice plate.	Plate is in place and works as designed.
	Obstructions	Any trash, debris, sediment, or vegetation blocking the plate.	Plate is free of all obstructions and works as designed.
Overflow Pipe	Obstructions	Any trash or debris blocking (or having the potential of blocking) the overflow pipe.	Pipe is free of all obstructions and works as designed.
Manhole	See "Closed Detention Systems" (No. 3).	See "Closed Detention Systems" (No. 3).	See "Closed Detention Systems" (No. 3).
Catch Basin	See "Catch Basins" (No. 5).	See "Catch Basins" (No. 5).	See "Catch Basins" (No. 5).

No. 5 – Catch Basins

Maintenance Component	Defect	Conditions When Maintenance is Needed	Results Expected When Maintenance is performed
General	Trash & Debris	Trash or debris which is located immediately in front of the catch basin opening or is blocking inletting capacity of the basin by more than 10%.	No Trash or debris located immediately in front of catch basin or on grate opening.
		Trash or debris (in the basin) that exceeds 60 percent of the sump depth as measured from the bottom of basin to invert of the lowest pipe into or out of the basin, but in no case less than a minimum of six inches clearance from the debris surface to the invert of the lowest pipe.	No trash or debris in the catch basin.

		Trash or debris in any inlet or outlet pipe blocking more than 1/3 of its height.	Inlet and outlet pipes free of trash or debris.
		Dead animals or vegetation that could generate odors that could cause complaints or dangerous gases (e.g., methane).	No dead animals or vegetation present within the catch basin.
	Sediment	Sediment (in the basin) that exceeds 60 percent of the sump depth as measured from the bottom of basin to invert of the lowest pipe into or out of the basin, but in no case less than a minimum of 6 inches clearance from the sediment surface to the invert of the lowest pipe.	No sediment in the catch basin
	Structure Damage to Frame and/or Top Slab	Top slab has holes larger than 2 square inches or cracks wider than 1/4 inch (Intent is to make sure no material is running into basin).	Top slab is free of holes and cracks.
		Frame not sitting flush on top slab, i.e., separation of more than 3/4 inch of the frame from the top slab. Frame not securely attached	Frame is sitting flush on the riser rings or top slab and firmly attached.
	Fractures or Cracks in Basin Walls/ Bottom	Maintenance person judges that structure is unsound.	Basin replaced or repaired to design standards.
		Grout fillet has separated or cracked wider than 1/2 inch and longer than 1 foot at the joint of any inlet/outlet pipe or any evidence of soil particles entering catch basin through cracks.	Pipe is regrouted and secure at basin wall.
	Settlement/ Misalignment	If failure of basin has created a safety, function, or design problem.	Basin replaced or repaired to design standards.
	Vegetation	Vegetation growing across and blocking more than 10% of the basin opening.	No vegetation blocking opening to basin.
		Vegetation growing in inlet/outlet pipe joints that is more than six inches tall and less than six inches apart.	No vegetation or root growth present.

Maintenance Component	Defect	Conditions When Maintenance is Needed	Results Expected When Maintenance is performed
	Contamination and Pollution	See "Detention Ponds" (No. 1).	No pollution present.
Catch Basin Cover	Cover Not in Place	Cover is missing or only partially in place. Any open catch basin requires maintenance.	Catch basin cover is closed
	Locking Mechanism Not Working	Mechanism cannot be opened by one maintenance person with proper tools. Bolts into frame have less than 1/2 inch of thread.	Mechanism opens with proper tools.
	Cover Difficult to Remove	One maintenance person cannot remove lid after applying normal lifting pressure. (Intent is keep cover from sealing off access to maintenance.)	Cover can be removed by one maintenance person.
Ladder	Ladder Rungs Unsafe	Ladder is unsafe due to missing rungs, not securely attached to basin wall, misalignment, rust, cracks, or sharp edges.	Ladder meets design standards and allows maintenance person safe access.
Metal Grates (If Applicable)	Grate opening Unsafe	Grate with opening wider than 7/8 inch.	Grate opening meets design standards.
	Trash and Debris	Trash and debris that is blocking more than 20% of grate surface inletting capacity.	Grate free of trash and debris.
	Damaged or Missing.	Grate missing or broken member(s) of the grate.	Grate is in place and meets design standards.

No. 5 – Catch Basins

Maintenance Components	Defect	Condition When Maintenance is Needed	Results Expected When Maintenance is Performed
General	Trash and Debris	Trash or debris that is plugging more than 20% of the openings in the barrier.	Barrier cleared to design flow capacity.
Metal	Damaged/ Missing Bars.	Bars are bent out of shape more than 3 inches.	Bars in place with no bends more than 3/4 inch.
		Bars are missing or entire barrier missing.	Bars in place according to design.
		Bars are loose and rust is causing 50% deterioration to any part of barrier.	Barrier replaced or repaired to design standards.
	Inlet/Outlet Pipe	Debris barrier missing or not attached to pipe	Barrier firmly attached to pipe

No. 7 – Energy Dissipaters

Maintenance Components	Defect	Conditions When Maintenance is Needed	Results Expected When Maintenance is Performed
External:			
Rock Pad	Missing or Moved Rock	Only one layer of rock exists above native soil in area five square feet or larger, or any exposure of native soil.	Rock pad replaced to design standards.
	Erosion	Soil erosion in or adjacent to rock pad.	Rock pad replaced to design standards.

Dispersion Trench	Pipe Plugged with Sediment	Accumulated sediment that exceeds 20% of the design depth.	Pipe cleaned/flushed so that it matches design.
	Not Discharging Water Properly	Visual evidence of water discharging at concentrated points along trench (normal condition is a "sheet flow" of water along trench). Intent is to prevent erosion damage.	Trench redesigned or rebuilt to standards.
	Perforations Plugged.	Over 1/2 of perforations in pipe are plugged with debris and sediment.	Perforated pipe cleaned or replaced.
	Water Flows Out Top of "Distributor" Catch Basin.	Maintenance person observes or receives credible report of water flowing out during any storm less than the design storm or its causing or appears likely to cause damage.	Facility rebuilt or redesigned to standards.
	Receiving Area Over-Saturated	Water in receiving area is causing or has potential of causing landslide problems.	No danger of landslides.
Internal:			
Manhole/Chamber	Worn or Damaged Post, Baffles, Side of Chamber	Structure dissipating flow deteriorates to 1/2 of original size or any concentrated worn spot exceeding one square foot which would make structure unsound.	Structure replaced to design standards.
	Other Defects	See "Catch Basins" (No. 5).	See "Catch Basins" (No. 5).

No. 11 – Wetponds

Maintenance Component	Defect	Condition When Maintenance is Needed	Results Expected When Maintenance is Performed
General	Water level	First cell is empty, doesn't hold water.	Line the first cell to maintain at least 4 feet of water. Although the second cell may drain, the first cell must remain full to control turbulence of the incoming flow and reduce sediment resuspension.
	Trash and Debris	Accumulation that exceeds 1 CF per 1000-SF of pond area.	Trash and debris removed from pond.
	Inlet/Outlet Pipe	Inlet/Outlet pipe clogged with sediment and/or debris material.	No clogging or blockage in the inlet and outlet piping.
	Sediment Accumulation in Pond Bottom	Sediment accumulations in pond bottom that exceeds the depth of sediment zone plus 6-inches, usually in the first cell.	Sediment removed from pond bottom.
	Oil Sheen on Water	Prevalent and visible oil sheen.	Oil removed from water using oil-absorbent pads or vactor truck. Source of oil located and corrected. If chronic low levels of oil persist, plant wetland plants such as Juncus effusus (soft rush) which can uptake small concentrations of oil.
	Erosion	Erosion of the pond's side slopes and/or scouring of the pond bottom, that exceeds 6-inches, or where continued erosion is prevalent.	Slopes stabilized using proper erosion control measures and repair methods.
	Settlement of Pond Dike/Berm	Any part of these components that has settled 4-inches or lower than the design elevation, or inspector determines dike/berm is unsound.	Dike/berm is repaired to specifications.
	Internal Berm	Berm dividing cells should be level.	Berm surface is leveled so that water flows evenly over entire length of berm.
	Overflow Spillway	Rock is missing and soil is exposed at top of spillway or outside slope.	Rocks replaced to specifications.

SECTION 12

SPECIAL REPORTS AND STUDIES

APPENDIX 12-A

GEOTECHNICAL REPORT

Associated Earth Sciences, Inc.

Serving the Pacific Northwest Since 1981

August 8, 2012
Project No. KE120280A

Select Homes, Inc.
16531 13th Avenue West, Suite A-107
Lynnwood, Washington 98037

Attention: Mr. Craig Pierce

Subject: Subsurface Exploration, Geologic Hazard, and Geotechnical Engineering Report
Eaglemont
Monroe, Washington

Dear Mr. Pierce:

We are pleased to present the enclosed copies of the above-referenced report. This report summarizes the results of our subsurface exploration, geologic hazard, and geotechnical engineering studies and offers recommendations for the preliminary design and development of the proposed project. Our recommendations are preliminary in that construction details have not been finalized at the time of this report.

We have enjoyed working with you on this study and are confident that the recommendations presented in this report will aid in the successful completion of your project. If you should have any questions or if we can be of additional help to you, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

Jon N. Sondergaard, L.G., L.E.G.
Senior Principal Geologist

JNS/pc
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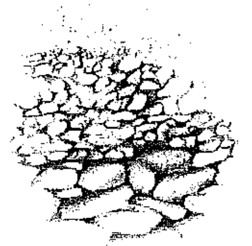

Geotechnical Engineering

Water Resources

Environmental Assessments and Remediation

Sustainable Development Services

Geologic Assessments

Associated Earth Sciences, Inc.

Serving the Pacific Northwest Since 1981

Subsurface Exploration, Geologic Hazard, and
Geotechnical Engineering Report

EAGLEMONT

Monroe, Washington

Prepared for

Select Homes, Inc.

Project No. KE120280A
August 8, 2012

SUBSURFACE EXPLORATION, GEOLOGIC HAZARD, AND GEOTECHNICAL ENGINEERING REPORT

EAGLEMONT

Monroe, Washington

Prepared for:
Select Homes, Inc.
16531 13th Avenue West, Suite A-107
Lynnwood, Washington 98037

Prepared by:
Associated Earth Sciences, Inc.
911 5th Avenue, Suite 100
Kirkland, Washington 98033
425-827-7701
Fax: 425-827-5424

August 8, 2012
Project No. KE120280A

I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of Associated Earth Sciences, Inc.'s (AESI's) subsurface exploration, geologic hazard, and geotechnical engineering study for Eaglemont, located on 197th Avenue SE off of Chain Lake Road in Monroe, Washington (Figure 1). The site boundaries, topographic contours, the proposed lot and road layout, and the approximate locations of the explorations accomplished for this study are presented on the "Site and Exploration Plan," Figure 2.

The recommendations in this report are considered to be preliminary because construction details were not finalized at the time of this study. Once development plans are substantially complete, the conclusions and recommendations in this report should be reviewed and modified, or verified, as appropriate.

1.1 Purpose and Scope

The purpose of this study was to provide subsurface data to be used in the preliminary design and development of the subject project. Our study included a review of available geologic literature, excavating seven exploration pits, and performing geologic studies to assess the type, thickness, distribution, and physical properties of the subsurface sediments and shallow ground water conditions. Geotechnical engineering studies were also conducted to assess the type of suitable foundation, allowable foundation soil bearing pressures, temporary cut slope recommendations, anticipated settlements, basement/retaining wall lateral pressures, floor support recommendations, and drainage recommendations. This report summarizes our current fieldwork and offers development recommendations based on our present understanding of the project.

1.2 Authorization

Written authorization to proceed with this study was granted by Mr. Randy Clark of Select Homes, Inc. Our study was accomplished in general accordance with our proposal dated July 6, 2012. This report has been prepared for the exclusive use of Select Homes, Inc., and their agents, for specific application to this project. Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering and engineering geology practices in effect in this area at the time our report was prepared. No other warranty, express or implied, is made.

2.0 PROJECT AND SITE DESCRIPTION

2.1 Site and Project Description

The subject site consists of an irregular-shaped parcel of approximately 35 acres. The property straddles 197th Avenue SE between Rainier View Road and Chain Lake Road in Monroe, Washington. The location of the subject site is shown on the "Vicinity Map," Figure 1. With the exception of a couple of extremely dilapidated, unoccupied buildings, the property is undeveloped and vegetated by mixed coniferous/deciduous forest with thick natural brush. The northern portion of the property is relatively flat-lying, but becomes gently to moderately sloping down toward the south in the southern portion of the site. Review of topographic contours shown on the attached "Site and Exploration Plan" indicate that slope inclinations in the southern portion of the site range from approximately 5 to 25 percent.

It is our understanding that project plans include subdividing the property into 149 residential parcels and constructing single-family homes on the lots with associated roads and utilities. The proposed lot and road layout is shown on the "Site and Exploration Plan," Figure 2.

3.0 SUBSURFACE EXPLORATION

Our field study included excavating a series of ten exploration pits to gain subsurface information about the site. The various types of sediments, as well as the depths where characteristics of the sediments changed, are indicated on the exploration logs presented in the Appendix. The depths indicated on the logs where conditions changed may represent gradational variations between sediment types. Our explorations were approximately located in the field relative to known site features shown on the attached site plan.

The conclusions and recommendations presented in this report are based, in part, on the exploration pits completed for this study. The number, locations, and depths of the explorations were completed within site and budgetary constraints. Because of the nature of exploratory work below ground, extrapolation of subsurface conditions between field explorations is necessary. Due to the random nature of deposition and the alteration of topography by past grading and/or filling, subsurface conditions may vary outside of the areas of the explorations. The nature and extent of any variations between the field explorations may not become fully evident until construction. If variations in subsurface conditions are observed at the time of construction, it may be necessary to re-evaluate specific recommendations in this report and make appropriate changes.

3.1 Exploration Pits

Exploration pits were excavated with a small track-mounted excavator. The pits permitted direct, visual observation of subsurface conditions. Materials encountered in the exploration pits were studied and classified in the field by an engineering geologist from our firm. Selected samples were then transported to our laboratory for further visual classification and testing, as necessary.

4.0 SUBSURFACE CONDITIONS

Subsurface conditions at the project site were inferred from the explorations completed for this study, our visual reconnaissance of the site, and review of applicable geologic literature. As shown on the exploration logs, the exploration pits generally encountered granular glacial sediments with high quantities of silt and moderate to high quantities of gravel. The following section presents more detailed subsurface information organized from the shallowest (youngest) to the deepest (oldest) sediment types.

4.1 Stratigraphy

Topsoil

An organic topsoil layer capped with either sod or forest duff was encountered at each of the exploration locations. The topsoil layer ranged in thickness from approximately 6 to 12 inches. Because of its relatively loose condition and high organic content, the topsoil is not considered suitable for foundation support or for use in a structural fill.

Vashon Lodgment Till

Sediments encountered directly below the topsoil layer at each of the exploration pit locations generally consisted of an unsorted mixture of loose to medium dense, reddish brown to tan, silty sand with gravel and scattered cobbles and boulders. Below depths ranging from approximately 2 to 4 feet, these sediments became dense to very dense and grayish tan. We interpret these sediments to be representative of Vashon lodgment till. The Vashon lodgment till consists of an unsorted mixture of silt, sand, and gravel that was deposited directly from basal, debris-laden glacial ice during the Vashon Stade of the Fraser Glaciation, approximately 12,500 to 15,000 years ago. The high relative density characteristic of the lodgment till is due to its consolidation by the massive weight of ice from which it was deposited. The reduced density and reddish brown to tan coloration observed in the upper portion of the till is interpreted to be due to weathering. At the locations of our explorations, the Vashon till extended beyond the maximum depths explored of approximately 5 to 6 feet.

Review of the regional geologic map of the area titled *Geologic Map of the Skykomish River 30- by 60-Minute Quadrangle, Washington*, compiled by Tabor, Frizzell, Booth, Waitt, Whetten, and Zartman (1993) indicates that the area of the project site is underlain by Vashon lodgment till. Our interpretation of the sediments encountered in our explorations is in agreement with the regional geologic map.

4.2 Hydrology

Thin zones of slow, perched, ground water seepage were encountered within the till at the locations of exploration pits EP-5 and EP-8 at depths of approximately 3 feet and 4 feet, respectively. At the locations of exploration pit EP-5, the seepage was present at the base of the weathered till horizon. At the location of exploration pit EP-8, the seepage was limited to a thin, sandy zone within the till at a depth of approximately 4 feet. It should be noted that the occurrence and level of ground water seepage at the site may vary in response to such factors as changes in season, amount of precipitation, and site use.

II. GEOLOGIC HAZARDS AND MITIGATIONS

The following discussion of potential geologic hazards is based on the geologic, slope, and shallow ground water conditions as observed and discussed herein and our review of the *City of Monroe Municipal Code* (MMC) for Critical Areas Title 20.05.

5.0 SEISMIC HAZARDS AND MITIGATIONS

Earthquakes occur in the Puget Lowland with great regularity. The vast majority of these events are small and are usually not felt by people. However, large earthquakes do occur, as evidenced by the 1949, 7.2-magnitude event; the 2001, 6.8-magnitude event; and the 1965, 6.5-magnitude event. The 1949 earthquake appears to have been the largest in this region during recorded history and was centered in the Olympia area. Evaluation of earthquake return rates indicates that an earthquake of the magnitude between 5.5 and 6.0 is likely in the Puget Sound area within a given 20-year period.

Generally, there are four types of potential geologic hazards associated with large seismic events: 1) surficial ground rupture, 2) seismically induced landslides, 3) liquefaction, and 4) ground motion. The potential for each of these hazards to adversely impact the proposed project is discussed below. In our opinion, the site is not a seismic hazard area according to MMC 20.05.

5.1 Surficial Ground Rupture

The nearest known fault traces to the project site are the South Whidbey Island Fault Zone (SWIFZ), located approximately 13 miles southwest of the site, and the Seattle Fault Zone, located approximately 19 miles to the south.

A 2005 study by the U.S. Geological Survey (USGS) (Sherrod, et al. 2005, *Holocene Fault Scarps and Shallow Magnetic Anomalies Along the Southern Whidbey Island Fault Zone near Woodinville, Washington*, Open-File Report 2005-1136, March 2005) reported that "strong" evidence of prehistoric earthquake activity has been observed along two fault strands thought to be part of the southeastward extension of the SWIFZ. The study suggests as many as nine earthquake events along the SWIFZ may have occurred within the last 16,400 years. The recognition of this fault splay is relatively new, and data pertaining to it are limited with the studies still ongoing. The recurrence interval of movement along this fault system is still unknown, although it is hypothesized to be in excess of one thousand years.

Studies of the Seattle Fault Zone by the USGS (e.g., Johnson, et al. 1994, *Origin and Evolution of the Seattle Fault and Seattle Basin, Washington*, Geology, v. 22, pp. 71-74; and Johnson, et al. 1999, *Active Tectonics of the Seattle Fault and Central Puget Sound*

Washington - Implications for Earthquake Hazards, Geological Society of America Bulletin, July 1999, v. 111, n. 7, pp. 1042-1053) have provided evidence of surficial ground rupture along a northern splay of the Seattle Fault. According to the USGS studies, the latest movement of this fault was about 1,100 years ago when about 20 feet of surficial displacement took place. This displacement can presently be seen in the form of raised, wave-cut beach terraces along Alki Point in West Seattle and Restoration Point at the south end of Bainbridge Island. The recurrence interval of movement along this fault system is still unknown, although it is hypothesized to be in excess of several thousand years.

Due to the suspected long recurrence intervals for both fault zones, the potential for surficial ground rupture is considered to be low during the expected life of the proposed structures.

5.2 Seismically Induced Landslides

It is our opinion that the risk of damage to the proposed structures by landsliding under both static and seismic conditions is low due to the lack of steep slopes on the subject site and adjoining areas. No mitigation of landslide hazards is warranted. In our opinion, the site is not a landslide hazard area according to MMC 20.05.

5.3 Liquefaction

It is our opinion that the sediments underlying the site present a low risk of liquefaction due their dense state and the lack of adverse ground water conditions. No mitigation of liquefaction hazards is warranted.

5.4 Ground Motion

Structural design of the building should follow 2009 *International Building Code* (IBC) standards using Site Class "C" as defined in Table 1613.5.2. The 2009 IBC seismic design parameters for short period (S_S) and 1-second period (S_1) spectral acceleration values were determined from the latitude and longitude of the project site using the USGS National Seismic Hazard Mapping Project website (http://earthquake.usgs.gov/hazmaps/). These values are based on Site Class "B". Based on the more current 2002 data, the USGS website interpolated ground motions at the project site to be 1.092g and 0.367g for building periods of 0.2 and 1.0 seconds, respectively, with a 2 percent chance of exceedance in 50 years. These values correspond to site coefficients F_a = 1.00 and F_v = 1.433, and a peak horizontal acceleration of 0.29g. The F_a, F_v, and peak horizontal acceleration values have been corrected for Site Class "C" in accordance with the IBC.

6.0 EROSION HAZARDS AND MITIGATIONS

The natural glacial sediments underlying the site generally contain a high percentage of silt and fine sand and are sensitive to erosion; however, the potential for erosion at the site is moderated by the fairly flat topography. In order to control erosion and reduce the amount of sediment transport off the site during construction, the following recommendations should be followed.

1. Properly embedded silt fencing should be placed around the lower perimeter of the cleared area(s). The fencing should be periodically inspected and maintained, as necessary, to ensure proper function.

2. The construction entrance should be stabilized with gravel pads to minimize tracking sediment off-site.

3. If possible, construction should proceed during the drier periods of the year.

4. Areas stripped of vegetation during construction should be mulched and hydroseeded, replanted as soon as possible, or otherwise protected. During winter construction, hydroseeded areas should be covered with clear plastic to facilitate grass growth.

5. If excavated soils are to be stockpiled on the site for reuse, measures should be taken to reduce the potential for erosion from the stockpile. These could include, but are not limited to, limiting stockpiled soil to the flatter areas of the site, covering stockpiles with plastic sheeting, and the use of straw bales/silt fences around pile perimeters.

Review of the U.S. Department of Agriculture Natural Resources Conservation Service (formerly known as the Soil Conservation Service) soil survey for the subject area, indicates that mapped soil types for the site include Tokul gravelly loam, 0 to 8 percent slopes, and Tokul gravelly loam 8 to 15 percent slopes. The mapped soil types are consistent with the sediments encountered in our explorations. Given presence of this soil type, the site does not classify as an erosion hazard area under MMC 20.05

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III. PRELIMINARY DESIGN RECOMMENDATIONS

7.0 INTRODUCTION

Our exploration indicates that, from a geotechnical standpoint, the parcel is suitable for the proposed development provided the recommendations contained herein are properly followed. The foundation bearing stratum is relatively shallow and conventional spread footing foundations may be utilized. Consequently, foundations bearing on either the medium dense to very dense, natural glacial sediments or on structural fill placed over these sediments are capable of providing suitable building support.

8.0 SITE PREPARATION

8.1 Clearing and Stripping

Following demolition of the existing structures, any underground utilities located within the proposed building areas should be removed or relocated. The resulting depressions should be backfilled with structural fill as discussed under the "Structural Fill" section of this report. Any remaining foundation elements that will not be incorporated into the new buildings should also be removed. Site preparation of the planned building areas should also include removal of all trees, brush, debris, and any other deleterious materials. These unsuitable materials should be properly disposed of off-site. Additionally, all organic topsoil within the proposed building areas, road areas, or areas to receive structural fill should be removed and the remaining roots grubbed. Areas where loose surficial soils exist due to grubbing operations should be considered as fill to the depth of disturbance and treated as subsequently recommended for structural fill placement. Any existing fill soils below footing areas should be stripped down to the underlying, medium dense to very dense natural till sediments. These sediments were encountered in our explorations at depths of approximately 1.5 to 3 feet.

8.2 Proof-Rolling

After stripping of the organic topsoil layer and removal of roots, we recommend that the soil exposed in proposed roadway areas be recompacted to a firm and unyielding condition using a 20-ton (minimum) vibratory roller. The recompacted area should then be proof-rolled with a fully loaded tandem-axle dump truck. Any soft or yielding areas identified during proof-rolling should be overexcavated and backfilled with structural fill.

8.3 Temporary and Permanent Cut Slopes

In our opinion, stable construction slopes should be the responsibility of the contractor and should be determined during construction based on the local conditions encountered at that time. For planning purposes, we anticipate that temporary, unsupported cut slopes in the loose to medium dense weathered native soils can be made at a maximum slope of 1.5H:1V (Horizontal:Vertical). Temporary cut slopes within the dense to very dense, unweathered till sediments can be planned up to a 1H:1V inclination. Flatter inclinations may be recommended in areas of seepage. In the dense to very dense till sediments, temporary vertical cuts no greater than 4 feet in height may also be constructed. As is typical with earthwork operations, some sloughing and raveling may occur, and cut slopes may have to be adjusted in the field. In addition, WISHA/OSHA regulations should be followed at all times.

Permanent cut or fill slopes should not exceed an inclination of 2H:1V.

8.4 Site Disturbance

The site soils contain a high percentage of fine-grained material, which makes them moisture-sensitive and subject to disturbance when wet. The contractor must use care during site preparation and excavation operations so that the underlying soils are not softened. If disturbance occurs, the softened soils should be removed and the area brought to grade with structural fill. If crushed rock is considered for the access and staging areas, it should be underlain by stabilization fabric (such as Mirafi 500X or approved equivalent) to reduce the potential of fine-grained materials pumping up through the rock and turning the area to mud. The fabric will also aid in supporting construction equipment, thus reducing the amount of crushed rock required. We recommend that at least 10 inches of rock be placed over the fabric; however, due to the variable nature of the near-surface soils and differences in wheel loads, this thickness may have to be adjusted by the contractor in the field. Crushed rock used for access and staging areas should be of at least 2-inch size.

9.0 STRUCTURAL FILL

Placement of structural fill may be necessary to establish desired grades in some areas. All references to structural fill in this report refer to subgrade preparation, fill type, and placement and compaction of materials as discussed in this section. If a percentage of compaction is specified under another section of this report, the value given in that section should be used.

9.1 Subgrade Compaction

After overexcavation/stripping has been performed to the satisfaction of the geotechnical engineer/engineering geologist, the upper 12 inches of exposed ground should be recompacted to a firm and unyielding condition. If the subgrade contains too much moisture, suitable recompaction may be difficult or impossible to attain and should probably not be attempted. In lieu of recompaction, the area to receive fill should be blanketed with washed rock or quarry spalls to act as a capillary break between the new fill and the wet subgrade. Where the exposed ground remains soft and further overexcavation is impractical, placement of an engineering stabilization fabric may be necessary to prevent contamination of the free-draining layer by silt migration from below. After the recompacted, exposed ground is tested and approved, or a free-draining rock course is laid, structural fill may be placed to attain desired grades.

9.2 Structural Fill Compaction

Structural fill is defined as non-organic soil, acceptable to the geotechnical engineer, placed in maximum 8-inch loose lifts, with each lift being compacted to at least 95 percent of the modified Proctor maximum dry density using *American Society for Testing and Materials* (ASTM):D 1557 as the standard. Roadway and utility trench backfill should be placed and compacted in accordance with applicable municipal codes and standards. The top of the compacted fill should extend horizontally a minimum distance of 3 feet beyond footings or pavement edges before sloping down at an angle no steeper than 2H:1V. Fill slopes should either be overbuilt and trimmed back to final grade or surface-compacted to the specified density.

9.3 Moisture-Sensitive Fill

Soils in which the amount of fine-grained material (smaller than No. 200 sieve) is greater than approximately 5 percent (measured on the minus No. 4 sieve size) should be considered moisture-sensitive. Use of moisture-sensitive soil in structural fills should be limited to favorable dry weather conditions. The on-site, natural glacial sediments are suitable for use as structural fill; however, they contain significant amounts of silt and are considered highly moisture-sensitive. At the time of our exploration, portions of the till sediments encountered in our exploration pits exhibited moisture contents in excess of the optimum for achieving maximum compaction. These soils are described on the attached exploration logs as "very moist" or "wet". These soils would require moisture conditioning prior to their use as structural fill. Such moisture conditioning could consist of spreading out and aerating the soil during periods of warm, dry weather.

Construction equipment traversing the site when the soils are very moist or wet can cause considerable disturbance. If fill is placed during wet weather or if proper compaction cannot be attained, a select import material consisting of a clean, free-draining gravel and/or sand should be used. Free-draining fill consists of non-organic soil with the amount of fine-grained material limited to 5 percent by weight when measured on the minus No. 4 sieve fraction.

9.4 Structural Fill Testing

The contractor should note that any proposed fill soils must be evaluated by AESI prior to their use in fills. This would require that we have a sample of the material at least 3 business days in advance to perform a Proctor test and determine its field compaction standard.

A representative from our firm should observe the stripped subgrade and be present during placement of structural fill to observe the work and perform a representative number of in-place density tests. In this way, the adequacy of the earthwork may be evaluated as filling progresses and any problem areas may be corrected at that time. It is important to understand that taking random compaction tests on a part-time basis will not assure uniformity or acceptable performance of a fill. As such, we are available to aid the owner in developing a suitable monitoring and testing frequency.

10.0 FOUNDATIONS

10.1 Allowable Soil Bearing Pressure

Spread footings may be used for building support when founded either directly on the medium dense to very dense, natural glacial sediments, or on structural fill placed over these materials. For footings founded either directly upon the medium dense to very dense glacial sediments, or on structural fill as described above, we recommend that an allowable bearing pressure of 2,000 pounds per square foot (psf) be used for design purposes, including both dead and live loads. For foundations founded totally upon dense to very dense unweathered till, a recommended allowable soil bearing pressure of 4,000 psf may be used. We recommend that the footing subgrade be recompacted to a firm and unyielding condition prior to footing placement. An increase in the allowable bearing pressure of one-third may be used for short-term wind or seismic loading. If structural fill is placed below footing areas, the structural fill should extend horizontally beyond the footing edges a distance equal to or greater than the thickness of the fill.

10.2 Footing Depths

Perimeter footings for the proposed buildings should be buried a minimum of 18 inches into the surrounding soil for frost protection. No minimum burial depth is required for interior footings; however, all footings must penetrate to the prescribed stratum, and no footings should be founded in or above loose, organic, or existing fill soils.

10.3 Footings Adjacent to Cuts

The area bounded by lines extending downward at 1H:1V from any footing must not intersect another footing or intersect a filled area that has not been compacted to at least 95 percent of ASTM:D 1557. In addition, a 1.5H:1V line extending down from any footing must not daylight because sloughing or raveling may eventually undermine the footing. Thus footings should not be placed near the edges of steps or cuts in the bearing soils.

10.4 Footing Settlement

Anticipated settlement of footings founded as described above should be on the order of 1 inch or less. However, disturbed soil not removed from footing excavations prior to footing placement could result in increased settlements.

10.5 Footing Subgrade Bearing Verification

All footing areas should be observed by AESI prior to placing concrete to verify that the exposed soils can support the design foundation bearing capacity and that construction conforms with the recommendations in this report. Foundation bearing verification may also be required by the governing municipality.

10.6 Foundation Drainage

Perimeter footing drains should be provided as discussed under the "Drainage Considerations" section of this report.

11.0 LATERAL WALL PRESSURES

All backfill behind walls or around foundations should be placed following our recommendations for structural fill and as described in this section of the report. Horizontally backfilled walls, which are free to yield laterally at least 0.1 percent of their height, may be designed using an equivalent fluid equal to 35 pounds per cubic foot (pcf). Fully restrained, horizontally backfilled, rigid walls that cannot yield should be designed for an equivalent fluid of 55 pcf. Walls that retain sloping backfill at a maximum angle of 50 percent should be designed for 45 pcf for yielding conditions and 65 pcf for restrained conditions. If parking areas or driveways are adjacent to walls, a surcharge equivalent to 2 feet of soil should be added to the wall height in determining lateral design forces.

11.1 Wall Backfill

The lateral pressures presented above are based on the conditions of a uniform backfill consisting of either the on-site glacial sediments or imported sand and gravel compacted to 90 to 95 percent of ASTM:D 1557. A higher degree of compaction is not recommended, as this will increase the pressure acting on the walls. A lower compaction may result in unacceptable settlement behind the walls. Thus, the compaction level is critical and must be tested by our firm during placement. The recommended compaction of 90 to 95 percent of ASTM:D 1557 applies to any structural fill placed behind the wall within a distance equal to the wall height and up to the elevation of the top of the wall. Structural fill used to construct slopes above retaining walls should be compacted to at least 95 percent of ASTM:D 1557 if the fill is placed above the elevation of the top of the wall. Surcharges from adjacent footings, heavy construction equipment, or sloping ground must be added to the above recommended lateral pressures. Footing drains should be provided for all retaining walls, as discussed under the "Drainage Considerations" section of this report.

11.2 Wall Drainage

It is imperative that proper drainage be provided so that hydrostatic pressures do not develop against the walls. This would involve installation of a minimum 1-foot-wide blanket drain for the full wall height using imported, washed gravel against the walls. If drainage mat is used it should be installed according to the manufacturer's specifications.

11.3 Passive Resistance and Friction Factor

Lateral loads can be resisted by friction between the foundation and the natural, medium dense to dense glacial sediments or supporting structural fill soils, or by passive earth pressure acting on the buried portions of the foundations. The foundations must be backfilled with compacted structural fill to achieve the passive resistance provided below. We recommend the following design parameters:

- Passive equivalent fluid = 250 pcf
- Coefficient of friction = 0.30

The above values are allowable.

11.4 Seismic Surcharge

As required by the 2009 IBC, retaining wall design should include a seismic surcharge pressure in addition to the equivalent fluid pressures presented above. Considering the site soils and the calculated peak horizontal acceleration of 0.29g, we recommend a seismic surcharge pressure of 9H to 12H where H is the wall height in feet for the "active" and "at-rest" loading conditions, respectively. The seismic surcharge should be modeled as a rectangular distribution with the resultant applied at the midpoint of the wall.

12.0 FLOOR SUPPORT

Slab-on-grade floors may be constructed either directly on the medium dense to very dense natural sediments, or on structural fill placed over these materials. Areas of the slab subgrade that are disturbed (loosened) during construction should be recompacted to an unyielding condition prior to placing the pea gravel, as described below.

If moisture intrusion through slab-on-grade floors is to be limited, the floors should be constructed atop a capillary break consisting of a minimum thickness of 4 inches of washed pea gravel, washed crushed rock, or other suitable material approved by the geotechnical engineer. The capillary break should be overlain by a 10-mil (minimum thickness) plastic vapor retarder.

13.0 DRAINAGE CONSIDERATIONS

The natural glacial sediments encountered in our explorations generally contained significant amounts of silt and are considered to be highly moisture-sensitive. Traffic from vehicles, construction equipment, and even foot traffic across these sediments when they are very moist or wet will result in disturbance of the otherwise firm stratum. Therefore, prior to site work and construction, the contractor should be prepared to provide drainage and subgrade protection, as necessary.

13.1 Wall/Foundation Drains

All retaining and perimeter footing walls should be provided with a drain at the footing elevation. The drains should consist of rigid, perforated, polyvinyl chloride (PVC) pipe surrounded by washed pea gravel. The level of the perforations in the pipe should be set approximately 2 inches below the bottom of the footing, and the drains should be constructed with sufficient gradient to allow gravity discharge away from the buildings. All retaining walls should be lined with a minimum, 12-inch-thick, washed gravel blanket provided to within 1 foot of finish grade, and which ties into the footing drain. If drainage mat is used it should be installed according to the manufacturer's specifications. Roof and surface runoff should not discharge into the footing drain system, but should be handled by a separate, rigid, tightline drain.

Exterior grades adjacent to walls should be sloped downward away from the structures to achieve surface drainage. Final exterior grades should promote free and positive drainage away from the buildings at all times. Water must not be allowed to pond or to collect adjacent to the foundation or within the immediate building area. It is recommended that a gradient of at least 3 percent for a minimum distance of 10 feet from the building perimeter be provided, except in paved locations. In paved locations, a minimum gradient of 1 percent should be provided unless provisions are included for collection and disposal of surface water adjacent to the structures. Additionally, pavement subgrades should be crowned to provide drainage toward catch basins and pavement edges.

14.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

We are available to provide additional geotechnical consultation as the project design develops and possibly changes from that upon which this report is based. If significant changes in grading are made, we recommend that AESI perform a geotechnical review of the plans prior to final design completion. In this way, our earthwork and foundation recommendations may be properly interpreted and implemented in the design. This plan review is not included in our current scope of work and budget.

We are also available to provide geotechnical engineering and monitoring services during construction. The integrity of the foundations depends on proper site preparation and construction procedures. In addition, engineering decisions may have to be made in the field in the event that variations in subsurface conditions become apparent. Construction monitoring services are not part of this current scope of work. If these services are desired, please let us know, and we will prepare a proposal.

We have enjoyed working with you on this study and are confident that these recommendations will aid in the successful completion of your project. If you should have any questions, or require further assistance, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

Timothy J. Peter, L.E.G., L.Hg.
Senior Project Geologist

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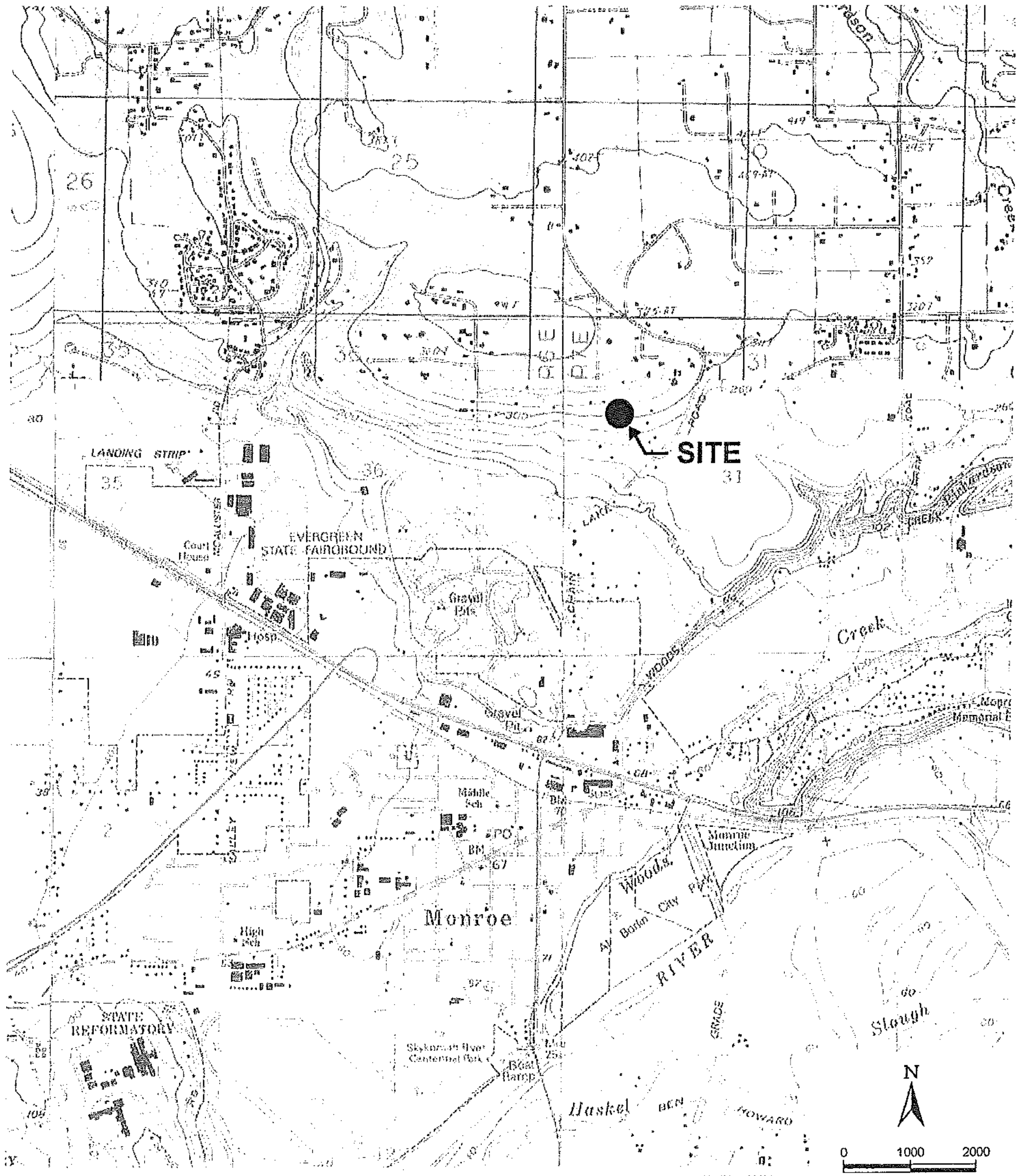
Senior Principal Geologist

Matthew A. Miller, P.E.
Principal Engineer

Attachments: Figure 1: Vicinity Map
Figure 2: Site and Exploration Plan
Appendix: Exploration Logs



REFERENCE: USGS TOPO!

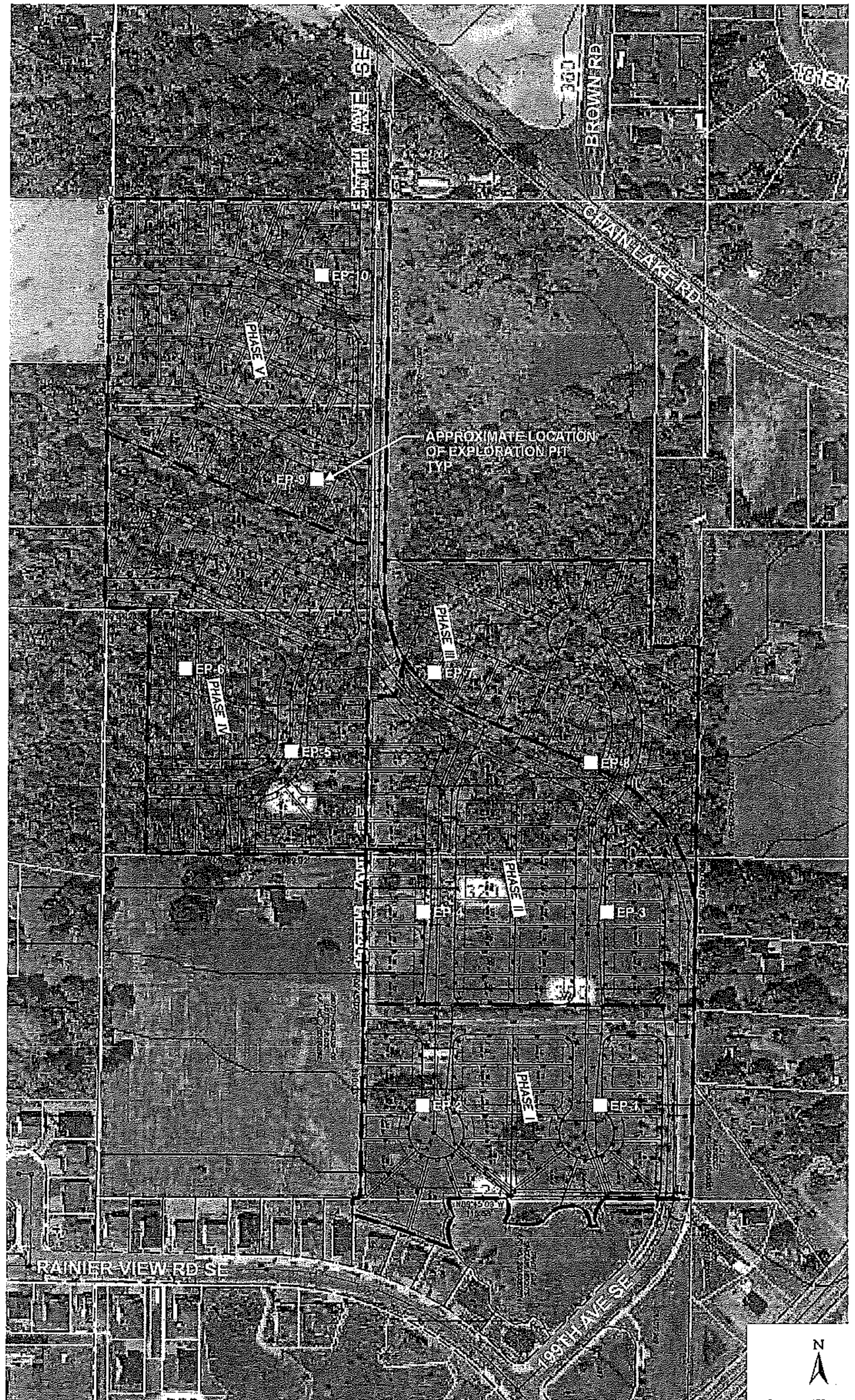
NOTE: BLACK AND WHITE REPRODUCTION OF THIS COLOR ORIGINAL MAY REDUCE ITS EFFECTIVENESS AND LEAD TO INCORRECT INTERPRETATION.

Associated Earth Sciences, Inc.

VICINITY MAP
EAGLEMONT
MONROE, WASHINGTON

FIGURE 1
DATE 7/12
PROJ. NO. KE120280A

Project Vicinity.cdr



REFERENCE: CLIENT

NOTE: BLACK AND WHITE REPRODUCTION OF THIS COLOR ORIGINAL MAY REDUCE ITS EFFECTIVENESS AND LEAD TO INCORRECT INTERPRETATION.

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SITE AND EXPLORATION PLAN
EAGLEMONT
MONROE, WASHINGTON

FIGURE 2
DATE 7/12
PROJ. NO. KE120280A

120280 Eaglemont \ 120280 Site and Expl.cdr

APPENDIX

Exploration Logs

Coarse-Grained Soils - More than 50% (1) Retained on No. 200 Sieve	Gravels - More than 50% (1) of Coarse Fraction Retained on No. 4 Sieve	≤5% Fines (5)	GW	Well-graded gravel and gravel with sand, little to no fines
			GP	Poorly-graded gravel and gravel with sand, little to no fines
		≥15% Fines (5)	GM	Silty gravel and silty gravel with sand
			GC	Clayey gravel and clayey gravel with sand
	Sands - 50% (1) or More of Coarse Fraction Passes No. 4 Sieve	≤5% Fines (5)	SW	Well-graded sand and sand with gravel, little to no fines
			SP	Poorly-graded sand and sand with gravel, little to no fines
		≥15% Fines (5)	SM	Silty sand and silty sand with gravel
			SC	Clayey sand and clayey sand with gravel
Fine-Grained Soils - 50% (1) or More Passes No. 200 Sieve	Silts and Clays Liquid Limit Less than 50		ML	Silt, sandy silt, gravelly silt, silt with sand or gravel
			CL	Clay of low to medium plasticity; silty, sandy, or gravelly clay, lean clay
			OL	Organic clay or silt of low plasticity
	Silts and Clays Liquid Limit 50 or More		MH	Elastic silt, clayey silt, silt with micaceous or diatomaceous fine sand or silt
			CH	Clay of high plasticity, sandy or gravelly clay, fat clay with sand or gravel
			OH	Organic clay or silt of medium to high plasticity
Highly Organic Soils			PT	Peat, muck and other highly organic soils

Terms Describing Relative Density and Consistency

	Density	SPT(2) blows/foot
Coarse-Grained Soils	Very Loose	0 to 4
	Loose	4 to 10
	Medium Dense	10 to 30
	Dense	30 to 50
	Very Dense	>50
	Consistency	**SPT(2) blows/foot**
Fine-Grained Soils	Very Soft	0 to 2
	Soft	2 to 4
	Medium Stiff	4 to 8
	Stiff	8 to 15
	Very Stiff	15 to 30
	Hard	>30

Test Symbols

G = Grain Size
M = Moisture Content
A = Atterberg Limits
C = Chemical
DD = Dry Density
K = Permeability

Component Definitions

Descriptive Term	Size Range and Sieve Number
Boulders	Larger than 12"
Cobbles	3" to 12"
Gravel	3" to No. 4 (4.75 mm)
Coarse Gravel	3" to 3/4"
Fine Gravel	3/4" to No. 4 (4.75 mm)
Sand	No. 4 (4.75 mm) to No. 200 (0.075 mm)
Coarse Sand	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	Smaller than No. 200 (0.075 mm)

(3) Estimated Percentage

Component	Percentage by Weight
Trace	<5
Few	5 to 10
Little	15 to 25
With	- Non-primary coarse constituents: ≥ 15% - Fines content between 5% and 15%

Moisture Content

Dry - Absence of moisture, dusty, dry to the touch
Slightly Moist - Perceptible moisture
Moist - Damp but no visible water
Very Moist - Water visible but not free draining
Wet - Visible free water, usually from below water table

Symbols

Sampler Type
Blows/6" or portion of 6"
2.0" OD Split-Spoon Sampler (SPT)
Bulk sample
Grab Sample

Sampler Type Description
3.0" OD Split-Spoon Sampler
3.25" OD Split-Spoon Ring Sampler
3.0" OD Thin-Wall Tube Sampler (including Shelby tube)
Portion not recovered

(1) Percentage by dry weight
(2) (SPT) Standard Penetration Test (ASTM D-1586)
(3) In General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)
(4) Depth of ground water
▼ ATD = At time of drilling
▽ Static water level (date)
(5) Combined USCS symbols used for fines between 5% and 15%

Classifications of soils in this report are based on visual field and/or laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual and/or laboratory classification methods of ASTM D-2487 and D-2488 were used as an identification guide for the Unified Soil Classification System.

LOG OF EXPLORATION PIT NO. EP-1

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. DESCRIPTION
	Sod / Topsoil
1	**Weathered Vashon Lodgement Till** Loose, moist, reddish brown, silty SAND, with gravel, scattered roots, scattered cobbles. Becomes medium dense and tan, with no roots below 1.5 feet.
2	
3	**Vashon Lodgement Till** Very dense, moist, grayish tan, silty SAND, with gravel (SM).
4	
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7	Bottom of exploration pit at depth 6 feet No seepage. No caving.
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KCTP3 120280.GPJ July 12, 2012

Eaglemont
Monroe, WA

Associated Earth Sciences, Inc.

Logged by: TJP
Approved by: TJP

Project No. KE120280A

7/10/12

LOG OF EXPLORATION PIT NO. EP-2

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. **DESCRIPTION**
	Sod / Topsoil
1	**Weathered Vashon Lodgement Till**
2	Loose, moist, reddish brown, silty SAND, with gravel (SM); scattered cobbles; abundant roots from 0 to 2 feet.
3	Becomes medium dense ant tan below 2.5 feet.
4	**Vashon Lodgement Till**
5	Very dense, moist, grayish tan, silty SAND, with gravel (SM).
6	
7	Bottom of exploration pit at depth 6 feet No seepage. No caving.
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Eaglemont
Monroe, WA

Associated Earth Sciences, Inc.

Logged by: TJP
Approved by: TJP

Project No. KE120280A
7/10/12

KCTP3 120280.GPJ July 24, 2012

LOG OF EXPLORATION PIT NO. EP-3

This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered.

Depth (ft)	DESCRIPTION
	Sod / Topsoil
1	**Weathered Vashon Lodgement Till**
2	Loose, moist, reddish brown, silty SAND, with gravel (SM).
3	Becomes medium dense and tan below 2.5 feet.
	Vashon Lodgement Till
4	Very dense, moist, grayish tan, silty SAND, with gravel, scattered cobbles.
5	
6	Bottom of exploration pit at depth 5.5 feet No seepage. No caving.
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Eaglemont
Monroe, WA

Associated Earth Sciences, Inc.

Logged by: TJP
Approved by: TJP

Project No. KE120280A
7/10/12

KCTP3 120280.GPJ July 24, 2012

LOG OF EXPLORATION PIT NO. EP-4

This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered.

Depth (ft)	DESCRIPTION
	Sod / Topsoil
1	**Weathered Vashon Lodgement Till**
2	Loose, moist, reddish brown, silty SAND, with gravel (SM). Becomes medium dense and tan below approximately 2 feet, scattered cobbles.
3	**Vashon Lodgement Till**
4	Very dense, moist, grayish tan, silty SAND, with gravel (SM); scattered cobbles.
5	
6	Bottom of exploration pit at depth 5 feet No seepage. No caving.
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Eaglemont
Monroe, WA

Associated Earth Sciences, Inc.

Logged by: TJP
Approved by: TJP

Project No. KE120280A
7/10/12

KCTP3 120280.GPJ July 12, 2012

LOG OF EXPLORATION PIT NO. EP-5

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. DESCRIPTION
	Forest Duff / Topsoil
	Weathered Vashon Lodgement Till
1	Loose, very moist, brown, silty SAND, with gravel, abundant roots (SM).
2	
3	Wet at base.
	Vashon Lodgement Till
4	Very dense, very moist, grayish tan, silty SAND, with gravel (SM). Becomes moist, contains scattered cobbles and boulders.
5	
6	Bottom of exploration pit at depth 5.5 feet Slow seepage at 3 feet. No caving.
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KCTP3 120280.GPJ July 12, 2012

Eaglemont
Monroe, WA

Associated Earth Sciences, Inc.

Logged by: TJP
Approved by: TJP

Project No. KE120280A

7/10/12

LOG OF EXPLORATION PIT NO. EP-6

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. **DESCRIPTION**
	Forest Duff / Topsoil
	Weathered Vashon Lodgement Till
1	Loose, very moist, reddish brown, silty SAND, with gravel, abundant roots (SM).
2	Wet at base.
	Vashon Lodgement Till
3	Very dense, very moist, grayish tan, silty SAND, with gravel, scattered cobbles (SM).
4	
5	
6	Bottom of exploration pit at depth 5.5 feet No seepage but sediments at base of weathered soil horizon (2.5 feet depth) appear close to saturated. No caving.
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LOG OF EXPLORATION PIT NO. EP-7

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. **DESCRIPTION**
	Forest Duff / Topsoil
1	**Weathered Vashon Lodgement Till** Loose, very moist, reddish brown, silty SAND, with gravel (SM).
2	Becomes very moist below 1.5 feet. Abundant roots from 0 to 2.5 feet.
3	**Vashon Lodgement Till** Dense to very dense, very moist, grayish tan, silty SAND, with gravel, scattered cobbles (SM).
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6	Bottom of exploration pit at depth 5.5 feet No seepage. No caving.
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KCTP3 120280.GPJ July 12, 2012

Eaglemont
Monroe, WA

Associated Earth Sciences, Inc.

Logged by: TJP
Approved by: TJP

Project No. KE120280A
7/10/12

LOG OF EXPLORATION PIT NO. EP-8

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. DESCRIPTION
	Forest Duff / Topsoil
	Weathered Vashon Lodgement Till
1	Loose, very moist, reddish brown, silty SAND, with gravel, scattered cobbles (SM).
	Abundant roots from 0 to 2 feet.
2	Becomes medium dense and tan below 2 feet.
	Vashon Lodgement Till
3	Very dense, very moist, grayish tan, silty SAND, with gravel, scattered cobbles (SM).
4	Becomes wet at approximately 4 feet.
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7	Bottom of exploration pit at depth 6.5 feet Slow seepage at 4 feet. No caving.
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Eaglemont
Monroe, WA

Associated Earth Sciences, Inc.

Logged by: TJP
Approved by: TJP

Project No. KE120280A
7/10/12

LOG OF EXPLORATION PIT NO. EP-9

This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered.

Depth (ft)	DESCRIPTION
	Forest Duff / Topsoil
1	**Weathered Vashon Lodgement Till**
	Loose, very moist, reddish brown, silty SAND, with gravel, scattered cobbles (SM).
2	Abundant roots from 0 to 2 feet.
	Becomes medium dense and tan below 2 feet.
3	**Vashon Lodgement Till**
	Very dense, very moist, grayish tan, silty SAND, with gravel, scattered cobbles, and boulders (SM).
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7	Bottom of exploration pit at depth 6 feet No seepage. No caving.
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KCTP3 120280.GPJ July 12, 2012

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LOG OF EXPLORATION PIT NO. EP-10

This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered.

Depth (ft)	DESCRIPTION
	Forest Duff / Topsoil
	Weathered Vashon Lodgement Till
1	Loose, moist to very moist, reddish brown, silty SAND, with gravel, abundant roots (SM).
2	**Vashon Lodgement Till**
3	Very dense, very moist, grayish tan, silty SAND, with gravel, scattered cobbles (SM).
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6	Bottom of exploration pit at depth 5.5 feet No seepage. No caving.
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KCTP3 120280.GPJ July 12, 2012

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